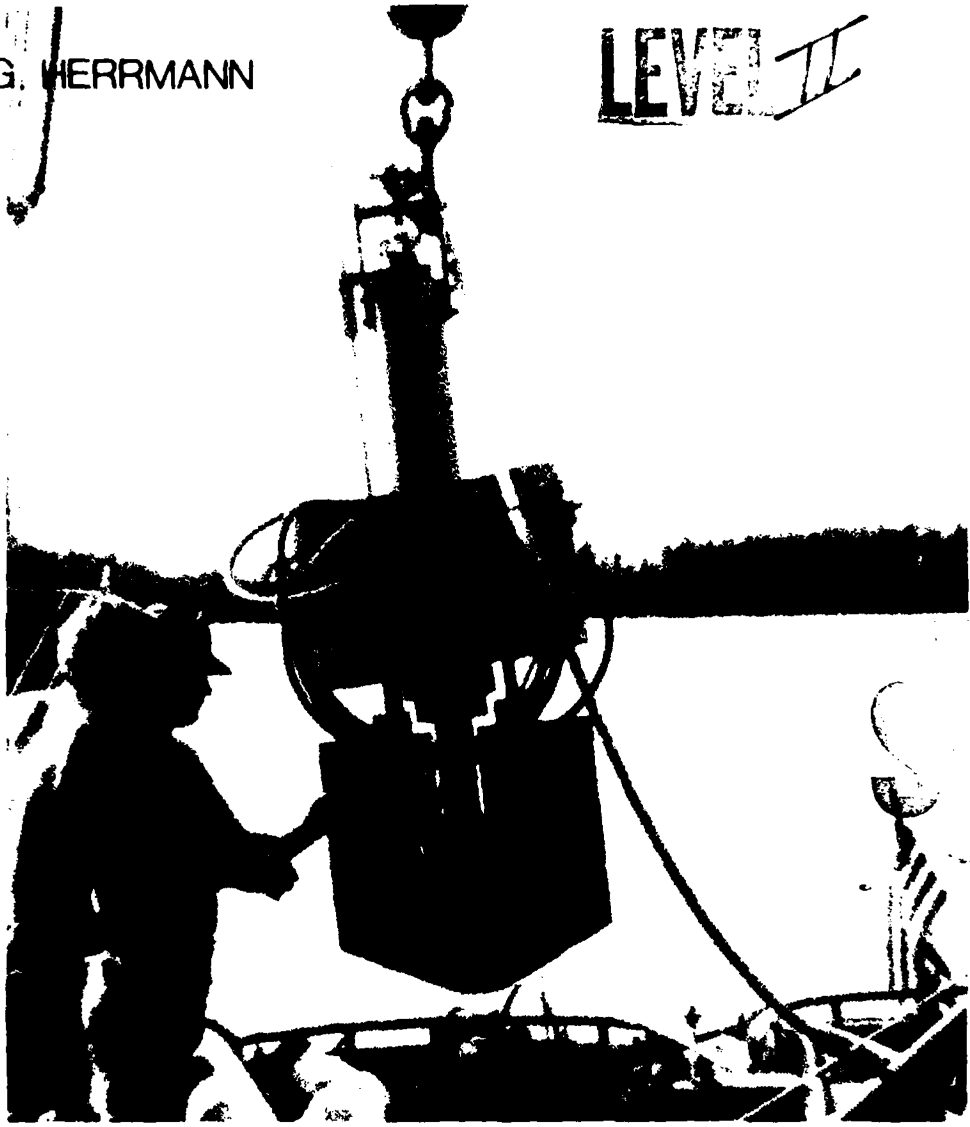
DESIGN PROCEDURES FOR EMBEDMENT ANCHORS SUBJECTED TO DYNAMIC LOADING CONDITIONS

BY H. G. HERRMANN



Sponsored by

NAVAL FACILITIES ENGINEERING COMMAND
Alexandria, Virginia 22332

NAVAL CIVIL ENGINEERING LABORATORY
Port Hueneme, California 93043

TECHNICAL REPORT R-888
NOVEMBER 1981

CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (force)	4.45	newtons
pounds (force) per square inch	6,894.757	pascals
pounds (force) per square foot	47.88026	pascals
feet per second	0.3048	meters per second
degrees (angle)	0.01745329	radians

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: TR-888	2. GOVT ACCESSION NO: DN787010
3. RECIPIENT'S CATALOG NUMBER: AD-A110 325	
4. TITLE (and Subtitle): DESIGN PROCEDURES FOR EMBEDMENT ANCHORS SUBJECTED TO DYNAMIC LOADING CONDITIONS	5. TYPE OF REPORT & PERIOD COVERED: Final; Jul 74 – Sep 78
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): H. G. Herrmann	8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS: NAVAL CIVIL ENGINEERING LABORATORY, Port Hueneme, California 93043	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS: 62759N; YF52.556.091.01.104
11. CONTROLLING OFFICE NAME AND ADDRESS: Naval Facilities Engineering Command, Alexandria, Virginia 22332	12. REPORT DATE: November 1981
	13. NUMBER OF PAGES: 70
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)	15. SECURITY CLASS. (of this report): Unclassified
	15a. DECLASSIFICATION DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Anchors, embedment, loads, mooring, site survey, design procedures.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides procedures for determining allowable design loads for embedment anchors under all types of dynamic loads, including impact (as from a ship driving into a mooring), cyclic (as from a buoy or ship riding in a mooring system during a storm), and earthquake loading. The design procedures are simplified in the form of guidelines applicable to most site conditions and are complete to the point of including the following: (1) site survey requirements; (2) detailed static design procedures; (3) description and definition of

continued

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. Continued

dynamic loads; (4) design procedures for impact, cyclic, and earthquake loading; (5) selection and use of appropriate factors of safety; and (6) description of available existing CEL embedment anchor hardware. While the procedures are designed specifically for the NCEL-developed family of propellant (often termed explosive) embedded anchors, they are applicable to other types of embedment anchors and screw anchors. For hardware configurations or site conditions beyond the range of the guidelines presented here, references to appropriate other documents are provided.

Library Card

Naval Civil Engineering Laboratory
DESIGN PROCEDURES FOR EMBEDMENT ANCHORS SUBJECTED TO DYNAMIC LOADING CONDITIONS (Final), by H. G. Herrmann
TR-888 70 pp illus November 1981 Unclassified

1. Embedment anchors 2. Design loads I. YF52.556.091.01.104

This report provides procedures for determining allowable design loads for embedment anchors under all types of dynamic loads, including impact (as from a ship driving into a mooring), cyclic (as from a buoy or ship riding in a mooring system during a storm), and earthquake loading. The design procedures are simplified in the form of guidelines applicable to most site conditions and are complete to the point of including the following: (1) site survey requirements; (2) detailed static design procedures; (3) description and definition of dynamic loads; (4) design procedures for impact, cyclic, and earthquake loading; (5) selection and use of appropriate factors of safety; and (6) description of available existing CEL embedment anchor hardware. While the procedures are designed specifically for the NCEL-developed family of propellant (often termed explosive) embedded anchors, they are applicable to other types of embedment anchors and screw anchors. For hardware configurations or site conditions beyond the range of the guidelines presented here, references to appropriate other documents are provided.

CONTENTS

	Page
INTRODUCTION	1
Scope	1
Summary of Anchor Behavior	2
Design Approach	3
RELATED DESIGN PROCEDURES AND REPORTS	3
Embedment Anchor Capacity	4
Ancillary Areas	5
Embedment Anchor Hardware	6
NCEL EMBEDMENT ANCHORS	7
DETERMINATION OF SITE DESIGN CONDITIONS	12
Recommended Site Survey	12
Check for Hazardous or Unusual Conditions	15
Procedures When Site Survey Is Limited	18
Variation of Soil Properties	20
Earthquakes	21
QUASI-STATIC HOLDING CAPACITY	21
Definitions and Line Angle Effect	21
Anchor Penetration Prediction	23
Anchor Keying	26
Anchor Verification	27
Holding Capacity Determination	30
Allowable Loads - Factor of Safety	33
DYNAMIC LOADING CONDITIONS	34
Impact Loads	35
Cyclic Loads	36
Earthquake Loading	42
DESIGN PROCEDURES FOR DYNAMIC/IMPACT LOADING	43
DESIGN PROCEDURES FOR CYCLIC/REPEATED LOADING CONDITIONS	47
Strength Loss During Anchor Cyclic Loading	48
Cyclic Creep During Anchor Cyclic Loading	50
Design for Earthquake Loading	52
Factors of Safety for Cyclic Loading	53
EFFECT OF LOAD HISTORY	54

ACKNOWLEDGMENTS 57

REFERENCES 59

LIST OF SYMBOLS 64

CONVERSION FACTORS, U. S. CUSTOMARY TO
METRIC (SI) UNITS OF MEASUREMENTS 67

INTRODUCTION

Scope

This report provides procedures for determining the design capacity of embedment anchors used in seafloor soils at any water depth (nearshore to very deep ocean) and subjected to loads ranging from short-term static (1 to 15-minute duration) to dynamic impact (durations as short as 0.01 second) and including repeated or cyclic loading as results from wave or ship motion, earthquake, and cable strumming. Proper design of embedment anchors is imperative because any significant overloading of this type of anchor can result in upward movement of the fluke which leads to reduced holding capacity and to the possibility of eventual failure and complete pullout.

These procedures were prepared for the Naval Civil Engineering Laboratory (NCEL) family of propellant-embedded anchors developed for Navy use; they are, however, applicable to other types of embedment anchors and screw anchors. For such other use, consideration should be given to the appropriateness of the disturbance factor f values, which are based on the NCEL embedment anchor configurations and method of operation.

The procedures outlined herein are necessarily generalized and simplified for broader application. To prevent the procedures from becoming overconservative (resulting in inefficient designs), their application has been necessarily limited to "normal" soil conditions. A number of soil types and conditions require more specialized analysis for proper anchor design. Procedures for identifying sites with these soils or conditions are provided, and reports are referenced which provide additional background and procedures for analyzing these more difficult situations.

This work was sponsored by the Naval Facilities Engineering Command under the Ocean Facilities Engineering Program.

Summary of Anchor Behavior

Embedment anchor capacity is a function of loading types as well as soil type and strength. Soils are divided into granular or cohesionless (sand) and cohesive (clay) types.

The most straightforward loading condition is termed "short-term quasi-static" and is often used as a basis or reference level for determining anchor capacity under other loading conditions. This condition corresponds to the undrained case for cohesive soils and the drained condition for granular soils.

A "long-term static-loading" condition implies that the soil is in drained equilibrium (fully consolidated); the time required to achieve the fully drained condition varies with the permeability of the soil – seconds to minutes for clean sand and months to years for highly plastic clays. Load capacity for cohesive soils under long-term static loading (as from a subsurface buoy) is generally higher than short-term static conditions except in creep-sensitive clays and heavily overconsolidated soils. Because of the short time required for drainage in sands, the short-term quasi-static condition is equivalent to the long-term drained condition.

Load capacity under dynamic impact loading is larger than for short-term static and gradually increases as the duration of the impact becomes shorter. For a series of repeated impact loads or cyclic loading conditions, there is a possibility of liquefying a granular soil or of causing remolding and strength reduction of a cohesive soil, both of which can result in significantly reduced anchor capacity compared to short-term static capacity. Loose granular soils are most susceptible to significant loss of strength.

Design Approach

This report summarizes pertinent background on loading conditions, related design studies and procedures, and Navy embedment anchors. Requirements and procedures are summarized for site evaluation, property determination, and checks for unusual or hazardous conditions that are beyond the scope of this report's design procedures. Typical conditions are described along with methods for estimating properties when site evaluation capabilities are limited. Methods for determining anchor capacity under short-term quasi-static loading conditions are presented along with procedures for predicting or measuring anchor keyed depth, which is an important input parameter. The nature of impact and repeated or cyclic loads are also described along with requirements and methods for describing these loads, including predictive procedures for several cases. The procedure for determining the increase in anchor capacity for impact-type loads is described. The design procedures for determining anchor load capacity under conditions of cyclic loading are provided along with an example problem. Also, the potential adverse effects of cyclic loading upon subsequent static or impact loading anchor capacity are summarized, as are the beneficial effect of time on strength recovery and potential increased resistance or strength under cyclic loading conditions. The effects of an earthquake on anchor performance are also outlined in quantitative terms.

RELATED DESIGN PROCEDURES AND REPORTS

A number of documents exist (or will be available shortly) which describe in more detail specific aspects of the generalized procedures presented here. These are listed and referenced below.

Embedment Anchor Capacity

Anchors Subjected to Impact Loading. Background information for the procedures summarized here is presented along with references to numerous other pertinent documents on the subject in:

Douglas, B. J. (1978). Effects of rapid loading rates on the holding capacity of direct embedment anchors, Civil Engineering Laboratory, P.O. Report No. 78-M-R420. Port Hueneme, Calif., Oct 1978.

Anchors Subjected to Cyclic Loading. Background information for the procedures summarized here, along with references to several other supporting studies, are presented in:

Gouda, Z. M., and D. G. True (1977). Dynamic loading effects on embedment anchor holding capacity – Interim report, Civil Engineering Laboratory, Technical Note No. N-1489. Port Hueneme, Calif., Jul 1977.

Static and Long-Term Holding Capacity. Background and design procedures with emphasis on long-term conditions are summarized in the following:

Beard, R. M. (1979). Long-term holding capacity of statically loaded anchors in cohesive soils, Civil Engineering Laboratory, Technical Note No. N-1545. Port Hueneme, Calif., Jan 1979.

Beard, R. M. (1980). Holding capacity of plate anchors, Civil Engineering Laboratory, Technical Report No. R-882. Port Hueneme, Calif., Oct 1980.

Holding Capacity in Rock. Embedment anchors using fluke configurations quite different from those used for muds and sands are quite effective in some seafloor rock types, as experience summarized in the following report indicates:

Wadsworth, J. F. (1976). *Anchoring in rock - A preliminary study*, Civil Engineering Laboratory, Technical Memorandum No. M-42-76-5. Port Hueneme, Calif., Apr 1976.

Research and development in this area is continuing.

Summary Reports. Procedures for holding capacity under various loading conditions based on the current state-of-the-knowledge are summarized in the following:

Beard, R. M. (1980). Holding capacity of plate anchors, Civil Engineering Laboratory, Technical Report No. R-882. Port Hueneme, Calif., Oct 1980.

The characteristics and performance of other types of uplift-resist ing anchors are summarized in the following handbook:

Taylor, R. J., D. Jones, and R. M. Beard (1975). Handbook for uplift-resisting anchors, Civil Engineering Laboratory. Port Hueneme, Calif., Sep 1975.

Ancillary Areas

Site Survey Techniques and Procedures. Methods for obtaining core samples and procedures for laboratory analysis of seafloor soils are summarized in:

Lee, H. J., and J. E. Clausner (1979). Seafloor soil sampling and geotechnical parameter determination - Handbook, Civil Engineering Laboratory, Technical Report No. R-873. Port Hueneme, Calif., Sep 1979.

General methods for soils analysis and classification together with terrestrial site survey techniques applicable to sheltered shallow water areas are summarized in the following:

Naval Facilities Engineering Command* (1971). Design manual -Soil mechanics, foundations, and earth structures, NAVFAC Design Manual DM-7. Washington, D.C., 1971.

The NCEL doppler penetrometer was developed as a tool for use in site survey work for embedment anchors in deeper water. Complete procedures for its use and interpretation of resulting data are summarized in the following:

Beard, R. M. (1976). Expendable doppler penetrometer: Interim report, Civil Engineering Laboratory, Technical Note No. N-1435. Port Hueneme, Calif., Apr 1976.

Beard, R. M. (1977). Expendable doppler penetrometer: A performance evaluation, Civil Engineering Laboratory, Technical Report No. R-855. Port Hueneme, Calif., Jul 1977.

Anchor Penetration and Verification. Complete background on the method of anchor penetration prediction used at NCEL, together with more sophisticated methods of analyses, are summarized in the following:

True, D. G. (1975). Penetration of projectiles into seafloor soils, Civil Engineering Laboratory, Technical Report No. R-822. Port Hueneme, Calif., May 1975.

The acoustic technique recommended here for determining anchor penetration and keying depth is described in more detail in the following document:

Malloy, R. J., and P. J. Valent (1978). Acoustic siting and verification of the holding capacity of embedment anchors, Civil Engineering Laboratory, Technical Note No. N-1523. Port Hueneme, Calif., Jul 1978.

Embedment Anchor Hardware

Descriptions of existing NCEL embedment anchors and their operating procedures are summarized in the following:

---

*NAVFAC.

Wadsworth, J. F., and R. J. Taylor (1976). CEL 10K propellant-actuated anchor, Civil Engineering Laboratory, Technical Note No. N-1441. Port Hueneme, Calif., Jun 1976.

Babineau, P. H., and R. J. Taylor (1976). *CEL 10K propellant-actuated anchor operations manual*, Civil Engineering Laboratory, Technical Memorandum No. M-42-76-3. Port Hueneme, Calif., Apr 1976.

Taylor, R. J. (1976). CEL 20K propellant-actuated anchor, Civil Engineering Laboratory, Technical Report No. R-837. Port Hueneme, Calif., Mar 1976.

Taylor, R. J., and P. H. Babineau (1974). Operation manual - Propellant-actuated deep water anchor, Civil Engineering Laboratory, Technical Memorandum No. M-42-74-1. Port Hueneme, Calif., Sep 1974.

NCEL EMBEDMENT ANCHORS

Descriptions of the four NCEL-developed embedment anchor systems currently available (or which will be shortly available in the case of the 300K anchor currently being tested) are summarized along with pertinent physical and typical performance characteristics in Table 1. The 10K anchor system is pictured in Figure 1 and displayed schematically in Figure 2. The other anchors are similar in appearance but larger in scale, as indicated by the information in Table 1. The operational sequence for anchor embedment and keying is illustrated in Figure 3. Typical values for fluke penetration and final keyed soil depth are listed in Table 1. The latter values assume a keying distance or travel of 1 to 1-1/2 fluke lengths in sand and from 1-1/2 to 2 fluke lengths in cohesive soils (muds). These low values were estimated and are applicable only to flukes designed in mid-1978 and later. These flukes utilize the wider fluke configuration for cohesive soils, the slightly larger keying arm lengths (see Figure 2), and a keying flap, consistent with the recommendations documented by Valent (1978). The final keyed soil depth D, as illustrated in Figure 3, has a major impact on anchor capacity. Methods for predicting, and then verifying, this depth are described in the section entitled Quasi-Static Holding Capacity.

Table 1. Nominal Characteristics of NCEL Embedment Anchors

Items and Characteristics	Measurements			
Anchor				
Anchor designation (nominal holding capacity) (lb)	10K	20K	100K	300K
Approximate total weight of anchor system (as pictured in Figure 1) (lb)	650	1,400	7,000	16,000
Sand Fluke				
Fluke length, L (ft)	2	3	5	7
Fluke width, B (ft)	1	2	2.5	4
Fluke projected area (for pullout), A (ft^2)	1.9	5.5	11	24
Weight of fluke and piston, W_T (lb)	160	290	1,300	4,200
Side area of fluke and piston (for penetration), A_s (ft^2)	6	14	40	62
Frontal area of fluke (for penetration), A_F (ft^2)	0.17	0.3	0.8	2
Length of fluke and piston, ℓ_T (ft)	4	5	8	12
Typical initial fluke velocity, v_o (fps)	385	460	500	520
Typical maximum tip penetration, D_{pk} (ft)	15	22	30	40
Typical minimum keyed depth, D (ft)	12	16	23	30
Mud Fluke				
Fluke length, L (ft)	2	3	6	8
Fluke width, B (ft)	2	3	4	7
Fluke projected area (for pullout), A (ft^2)	3.7	8.5	28	56
Weight of fluke and piston, W_T (lb)	185	420	2,100	6,800
Side area of fluke and piston (for penetration), A_s (ft^2)	10	21	80	145
Frontal area of fluke (for penetration), A_F (ft^2)	0.22	0.4	1.2	3
Length of fluke and piston, ℓ_T (ft)	4	5	9	13
Typical initial fluke velocity, v_o (fps)	370	360	380	380
Typical maximum tip penetration, D_{pk} (ft)	30	35	55	75
Typical minimum keyed depth, D (ft)	26	30	43	60

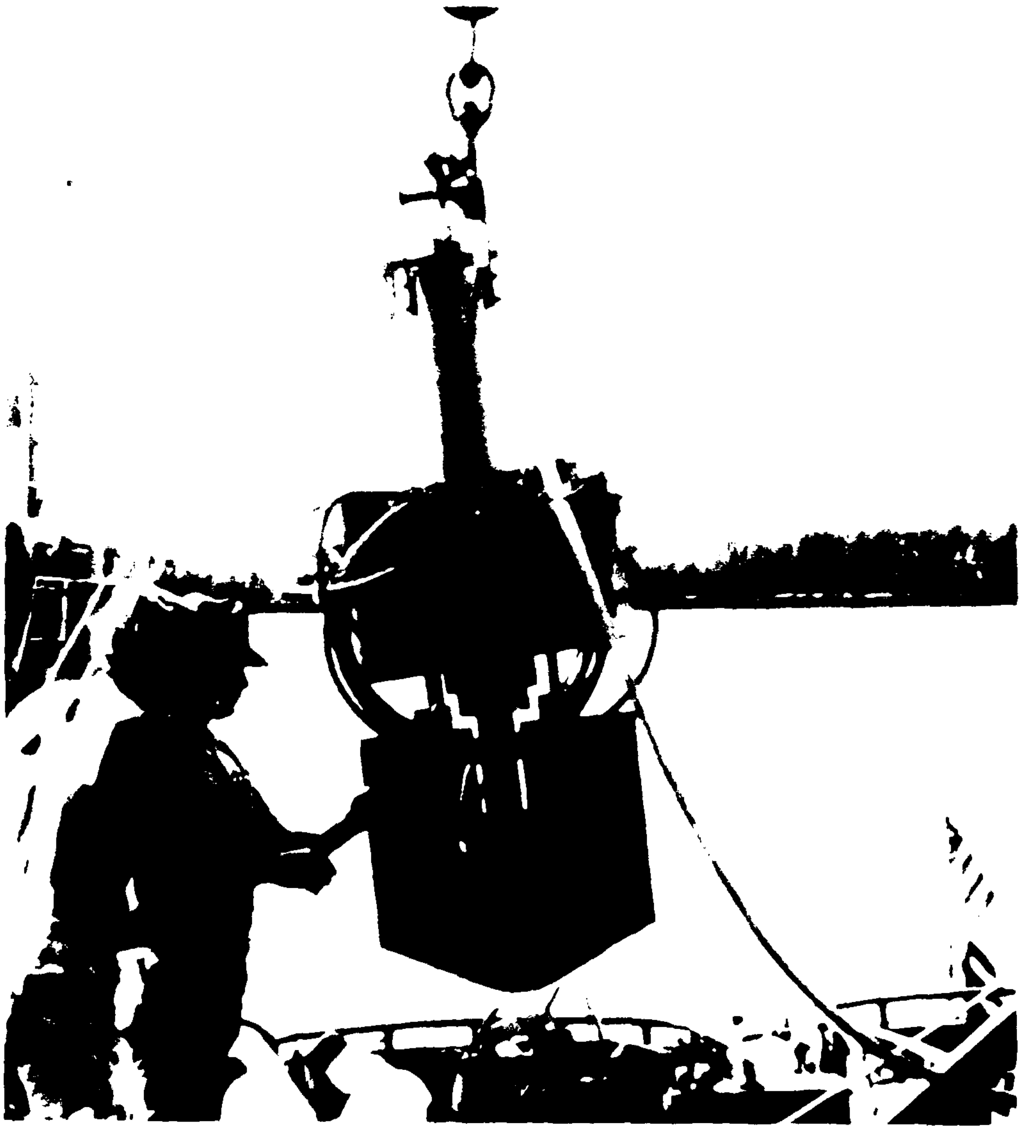


Figure 1. NCEL 10K propellant-embedded anchor with mud fluke.

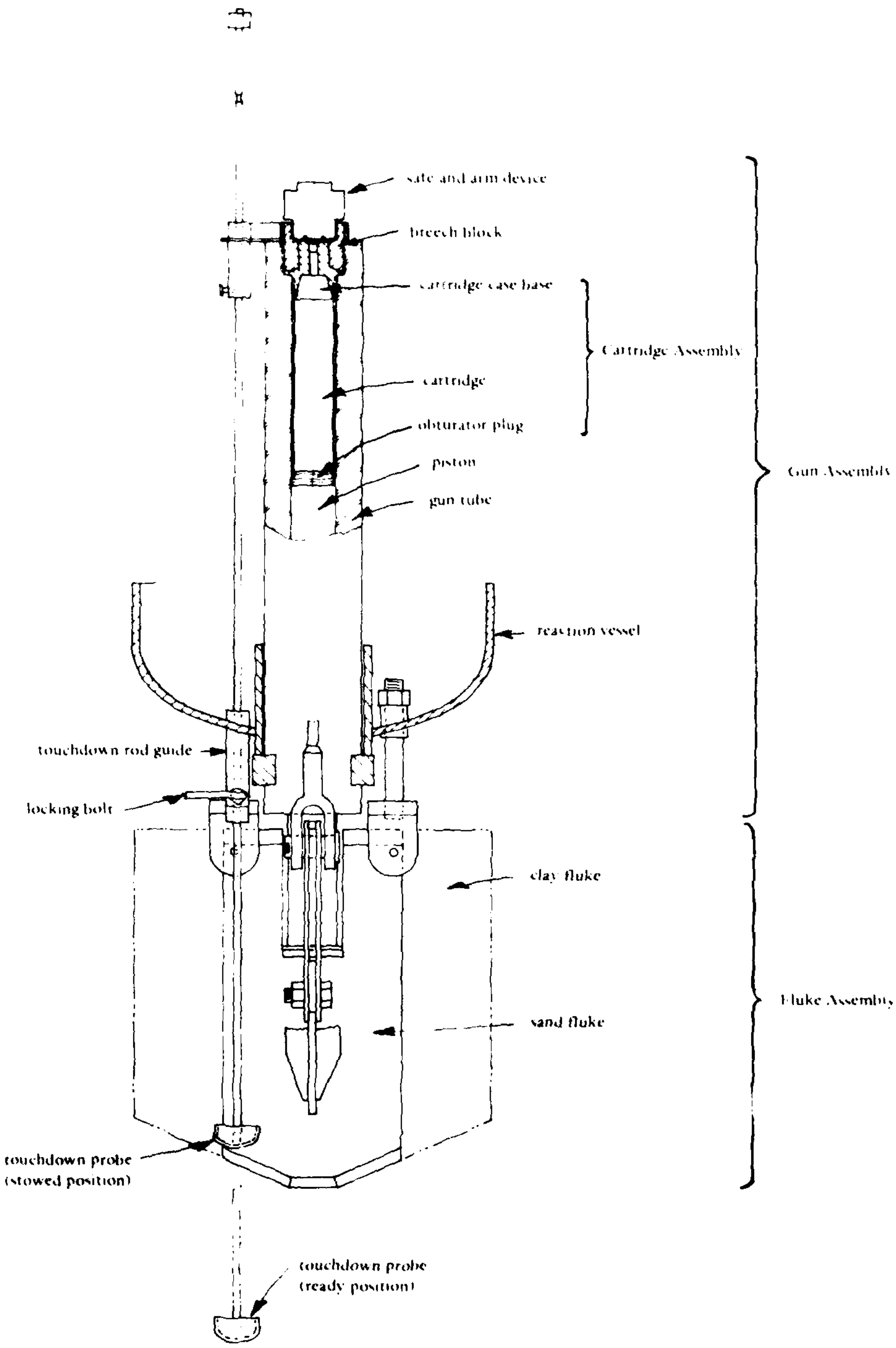


Figure 2. Schematic of the NCEL 10K propellant-embedded anchor.

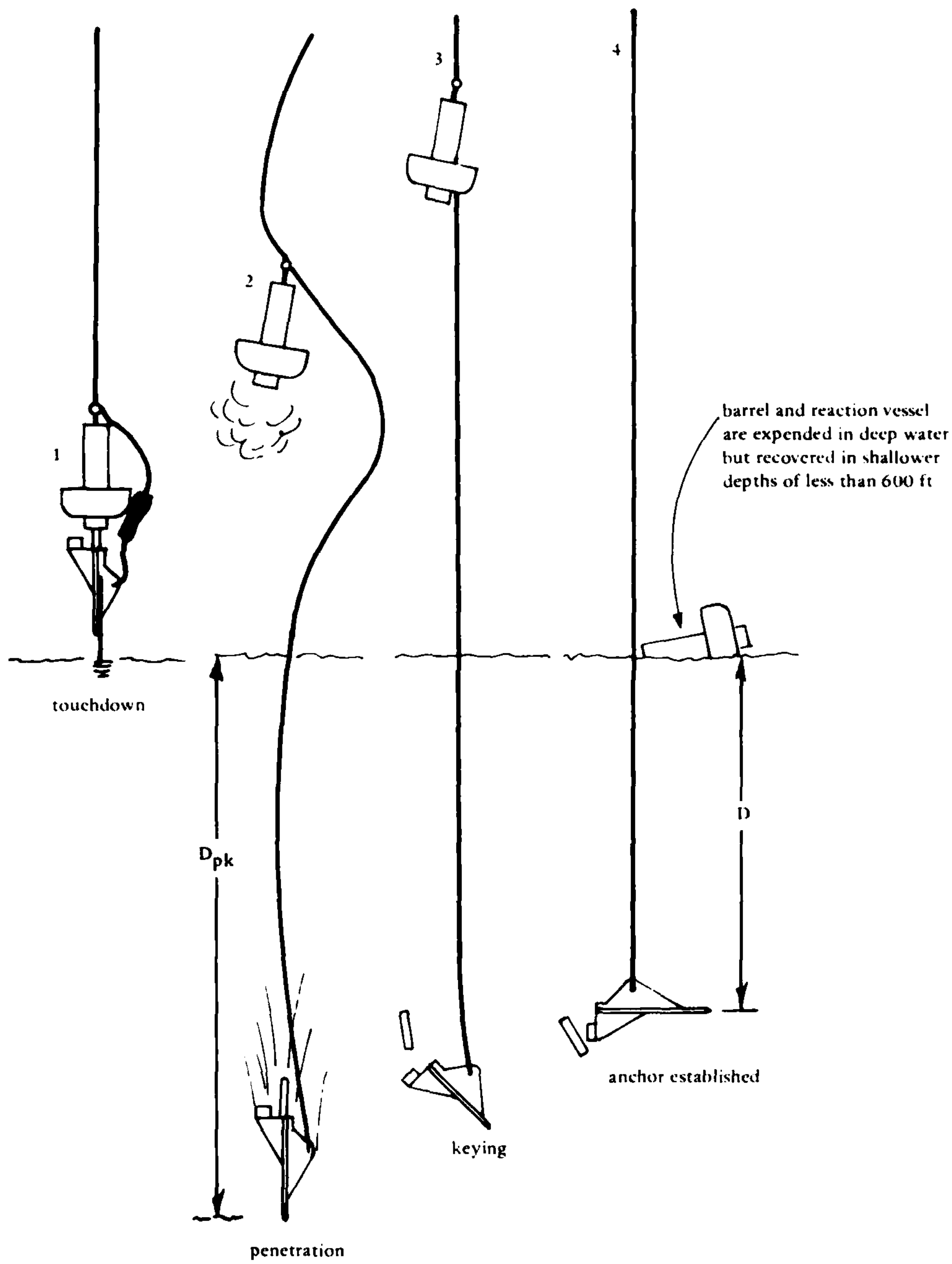


Figure 3. Schematic of typical sequence for anchor embedment.

DETERMINATION OF SITE DESIGN CONDITIONS

Recommended Site Survey

There are two recommended deep water site survey methods. Either will provide satisfactory data.

(1) A relatively undisturbed core sample at least as long as the expected depth of anchor penetration

(2) A core sample of the upper 6 to 8 feet of soil together with data from an NCEL doppler penetrometer or similar device providing comparable data and depths of penetration

The survey should be performed close to the proposed anchor location so that the soil samples will provide representative and reliable data.

At shallower water sites it may be less expensive, and even technically preferable, to use adaptations of terrestrial site survey techniques that provide data similar to those required for deep water locations. Reliable sampling of, or measurements on, granular soils (sands) are difficult to achieve. If a highly reliable design is required, penetration tests with dynamic penetrometers (e.g., the doppler penetrometer) or impact types (e.g., Standard Penetration Test described in Naval Facilities Engineering Command, 1971) are recommended. If the latter is used, the friction angle, ϕ', can be determined from the blow count record by use of the graph in Figure 4.

The core samples should be analyzed along with the results from any penetrometer tests to determine the following characteristics and the samples' general variation with soil depth:

(1) Soil grain size. Soil type is classified based on grain size and plasticity. Classification as granular (sand) or cohesive (muds and clays) is mandatory; classification by a system such as the Unified Soil Classification System (see Naval Facilities Engineering Command, 1971) is preferable.

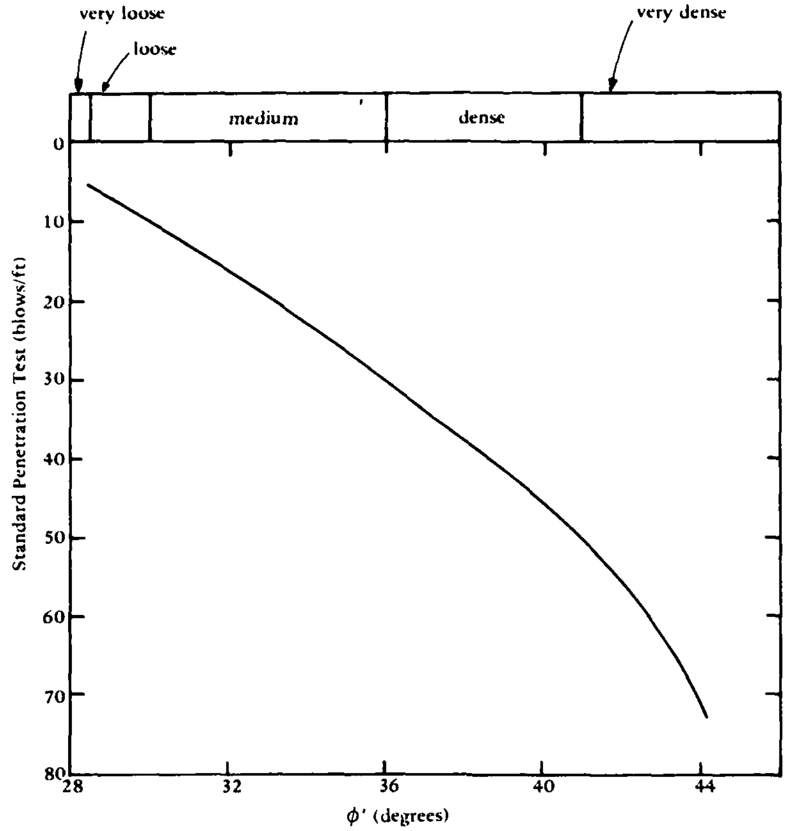


Figure 4. Relationship of standard penetration test blow count to angle of internal friction (from Peck, Hanson, and Thornburn, 1953. Used by permission of John Wiley & Sons, Inc.).

(2) Soil origin. Deep ocean soils should be classified by percentage of that portion of their dry weight that is biogenic in origin (calcareous or siliceous oozes). This can be done by microscopic or chemical analysis (see Lee and Clausner, 1979).

(3) Soil strength. For cohesive soils, the undrained shear strength, s_u, should be obtained using a vane shear device (at a rotation speed of 6 deg/min or less) or in laboratory undrained shear tests (direct or, preferably, triaxial shear - especially for soils exhibiting any unusual behavioral patterns). The remolded strength should also be measured (easiest to accomplish with the vane shear device), and the soil's sensitivity, S_T, evaluated (S_T = ratio of undisturbed to remolded strength). For granular soils, the drained friction angle, ϕ', should be measured. It is difficult to obtain a truly representative sample or to prepare one in the laboratory for evaluating ϕ'. However,

since the value of ϕ_u', not a peak value, is needed for analysis of holding capacity, a slightly disturbed and less dense sample will not result in any significant error for the site conditions to which this report is applicable.

(4) Soil density. Density in terms of the submerged or buoyant unit weight, γ_b, should be measured.

(5) Soil plasticity. The plasticity of the soil should be measured in the laboratory to determine the soil's Plasticity Index (PI).

Ranges for, and typical values of, several of the above properties are given in Table 2 for several categories of typical seafloor types. The strength of the soil at a site may change over time as a result of the installation of an anchor and also as a result of the various types of loads to which the soil is subsequently subjected. These changes and their quantification are discussed in the section entitled Effect of Load History.

Table 2. Ranges and Typical Values of Seafloor Soils Properties (Based on data from a variety of sources, including Horn et al. (1974), Hough (1969), Keller (1974), and McClelland Engineers (1976))

Soil	S_T	ϕ' (deg)	γ_b (pcf)	PI
Continental Shelf Clays	1-4	--	8-103, 30[a]	N.P.[b]-70
Continental Margin and Deep Ocean Clays	1-88, 4[a]	--	12-67, 32[a]	15-120
Calcareous Ooze	5-12	N.A.[c]-37	18-62, 34[a]	N.P.-110
Siliceous Ooze	High	Low	4-29, 16[a]	N.P.-127
Beach-like Sands	--	30-36, 32[a]	52-73, 63[a]	N.P.
Silty Sands	--	25-40, 30[a]	54-79, 65[a]	N.P.

[a]Typical value.

[b]N.P. - Nonplastic.

[c]N.A. - Not applicable.

Check for Hazardous or Unusual Conditions

Since the guidelines presented here are a simplified procedure based on "ordinary" conditions, it is necessary to check for hazardous or unusual conditions for which these guidelines would not be satisfactory. A satisfactory design would still be possible, but it would require more sophisticated analyses based on more detailed procedures, such as those referenced in an earlier section entitled Related Design Procedures and Reports.

The following checks should be made of the soil profile to the soil depth of expected maximum anchor penetration. If any are positive, the site conditions are too unusual for the guidelines provided here, and application would result in possibly unsafe designs.

(1) Are either bedrock or pieces of rock larger than gravel size (2 inches in diameter) present in the soil profile at soil depths less than the maximum expected fluke tip penetration?

(2) Does the soil type change significantly, or are there major layers of different soil classes (e.g., 10 feet of mud overlying sand, a 3-foot-thick layer of sand 10 feet deep in a clay profile, or numerous turbidite sand layers in a deep ocean clay profile)?

(3) Is the soil from a deep ocean site either a siliceous ooze (defined as being 30% by weight biogenic in origin and siliceous in make-up; i.e., made up of the shells of diatoms or radiolarians and characterized by very high void ratios - values of 6 to 8 are common) or a "clean" calcareous ooze (defined here as being at least 60% by weight biogenic in origin and calcareous in make-up)? The general regions where these types of sediments tend to occur are illustrated in Figure 5. While major areas of the seafloor are made up of calcareous ooze, only a small percentage of this area would be classified as a "clean" calcareous ooze with the more troublesome behavior characteristics. The regular calcareous ooze contains a larger percentage of clay

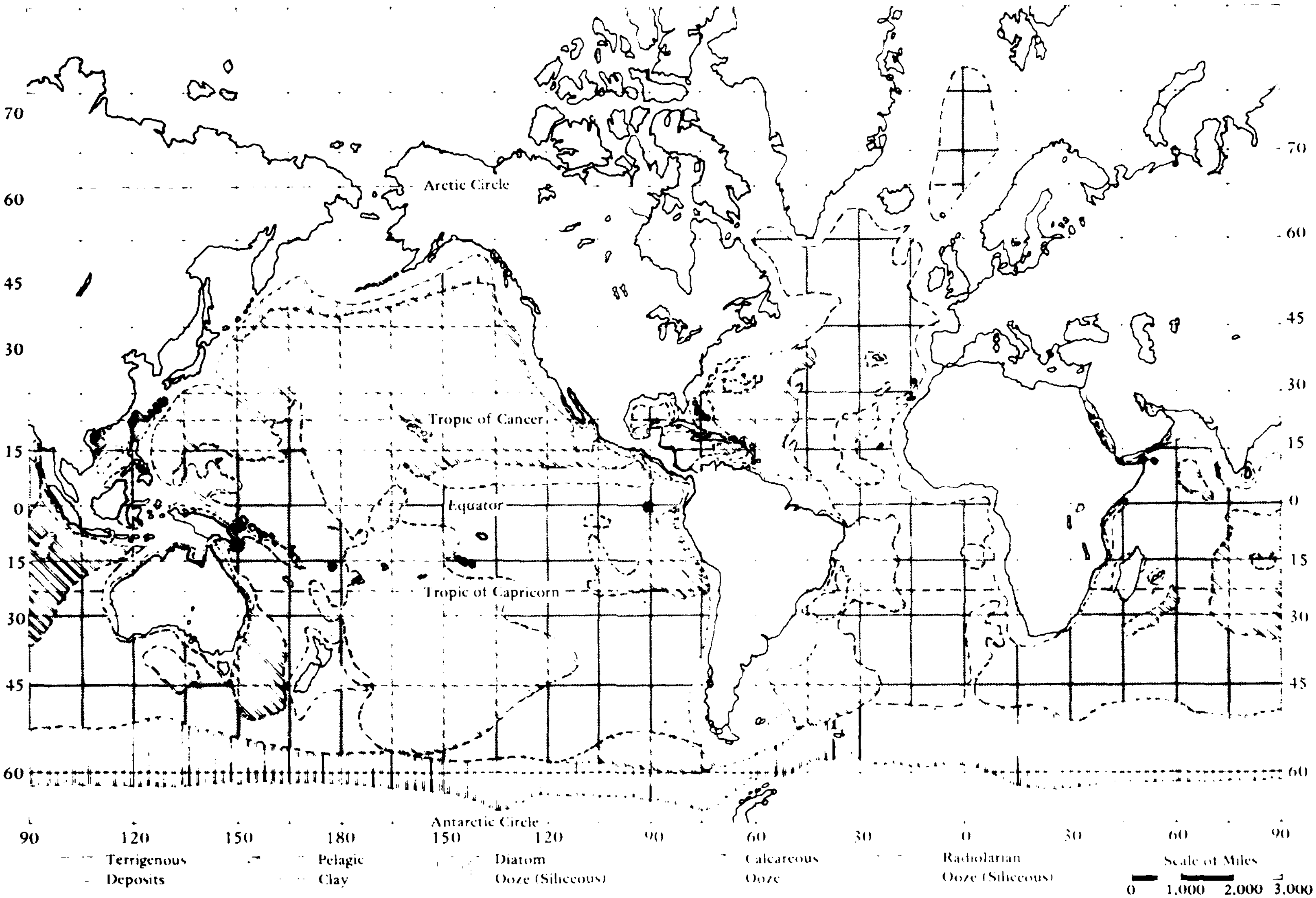


Figure 5. Distribution of soil types in the oceans of the world (from Herrmann et al., 1972).

minerals and as a result behaves more like a cohesive soil with behavior characteristics within the range for which these guidelines were developed.

(4) Does the cohesive soil exhibit high sensitivity (i.e., $S_T > 6$)?

(5) Does the cohesive soil exhibit other than a normal soil profile of either constant or gradually increasing strength with soil depth? A normal rate of strength increase with depth can be determined by reference to Figure 6. With the soil's PI, a reasonable value of the strength/plasticity (c/p) ratio can be estimated. Then with this ratio and the buoyant unit weight, γ_b, of the soil, a normal strength profile can be estimated. Does the measured soil profile consistently differ from the estimated normal profile by more than -50% or +100% in other than the upper few feet?

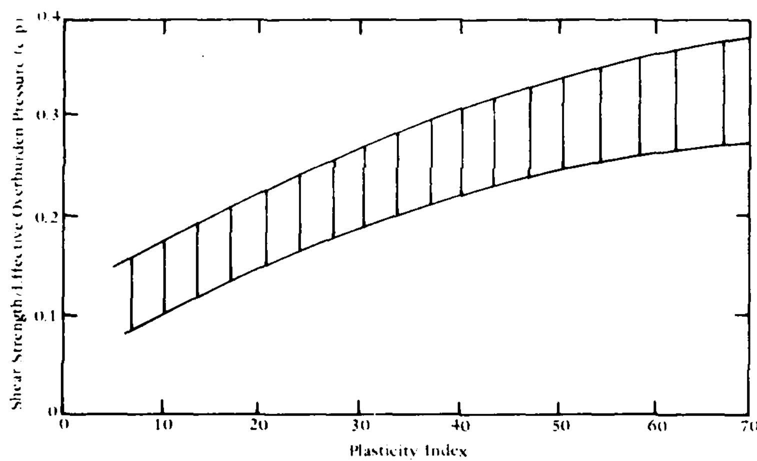


Figure 6. Relationship of c/p ratio to plasticity index (from Handbook of Ocean and Underwater Engineering by B. McClelland. © 1969. Used with permission of McGraw Hill Book Company).

(6) Is the cohesive soil consistently overconsolidated (in other than the upper few feet) by a factor of at least 2 (an overconsolidation ratio, OCR, greater than 2)? This will show up in item (5) as a strength well above that of a normal profile.

(7) Is the site located on a slope greater than 10 degrees?

If any of the above checks are positive, indicating an unusual or more troublesome condition, the anchor capacity cannot be reliably estimated using the simplified procedures in the sections which follow. A background for dealing with a troublesome site can be gained by reference to reports cited in the earlier section entitled Related Design Procedures and Reports, or to Herrmann and Houston (1976), Herrmann and Houston (1978), or Houston and Herrmann (1980). However, designing for such sites will probably require consultation with an expert.

Procedures When Site Survey Is Limited

The highly recommended procedures for proper site survey outlined earlier cannot always be followed or comparable techniques employed. Accurate evaluation of granular soil properties is particularly difficult even with the recommended procedures. Fortunately, the excellent performance record of embedment anchors in sand indicates that a practical design will result if the properties of sands are estimated in a reasonable and conservative manner as outlined below. In the case of cohesive soils, estimating properties for such sites introduces a larger margin for potential error because of the wide range of anchor performance that can result. Estimates should only be made for cohesive soils where the consequences of a failure are not major; even in these cases an increased factor of safety F_s is recommended to account for potential errors in these estimates.

Estimates for Granular Soils. The values listed below may be used as conservative estimates for the types of nonplastic granular soils indicated.

Soil Type	ϕ' (deg)	γ_b (pcf)
Sandy silt	20	60
Silty sand	25	60
Uniform sand	30*	55
Well-graded sand	33*	60

*Four degrees may be added to each of these values if visual inspection of the sand indicates that the particles are angular. Beach sands are always rounded; an angular sand will have sharp edges and corners as revealed under a microscope or magnifying glass.

Use of these values in the design procedures for sites with the indicated soil types will result in conservative designs which require no additional F_s - only that applied in the normal design procedure.

Estimates for Cohesive Soils. At sites where only short core samples can be obtained, properties can be extrapolated with soil depth, using the approach outlined in item (5) of the preceding section entitled Check for Hazardous or Unusual Conditions. The accuracy of that extrapolation can be validated during anchor installation by monitoring anchor embedment and proper seating under keying/proof loading - checks that are recommended for all anchor installations; this procedure is discussed in detail in the subsection entitled Anchor Verification which follows.

If no data are available for a site (a disturbed grab sample is much better than no data at all) and if there is no reason to suspect unusual conditions such as the following:

(1) siliceous or calcareous oozes in deep water

(2) rock (which is easily detected on subbottom profiles and common in areas of rugged topography)

(3) underconsolidated weak soils due to rapid sedimentation (such as occurs in the vicinity of river mouths)

(4) overconsolidation which can occur in water depths less than 400 feet due to past geological changes such as lower sea level stands

(5) irregular soil profiles which are more common in continental shelf regions

then it is best to assume a normal profile of low plasticity cohesive soil (with a c/p ratio of 0.10 and a γ_b of 30 pcf). The behavior of the anchor during installation and keying/proof loading should be monitored carefully, and a larger F_s is suggested (total F_s value of 3). This approach (assuming the above soil profile in the absence of all data) should not be used for anchors in critical applications.

Variation of Soil Properties

Disturbance Due to Anchor Penetration. The physical penetration of the anchor fluke into the soil and its motion during keying cause some mechanical disturbance or remolding of the soil, which results in a reduction of soil strength. This reduction from the original values determined during the site survey phase is handled quantitatively by the disturbance factor, f, which is applied to all short-term static and dynamic holding capacity determinations on cohesive soils. An f-value of 0.7 is currently recommended for the nonhazardous soils addressed in this design procedure. Future work is expected to better define this parameter and its variation with soil type or sensitivity.

Strength Increase Due Consolidation. The cohesive soil around an anchor will drain or consolidate under the influence of long-term static loads applied by the anchor. Whether or not a long-term static load is applied, the disturbed zone described in the preceding subsection will regain much of its lost strength. If a uniform static load (as from a submerged buoy) is applied and is at a safe long-term load level (see Beard, 1979, to make such a determination), the soil strength will increase for most site conditions.

Because the magnitude and rate of both of the strength increases mentioned are difficult to determine precisely and generally require more sophisticated laboratory testing and soil analysis, this strength increase effect is not included quantitatively in the design procedures which follow. From a practical standpoint, it would be unusual to have an anchor used in a situation where it was subjected to a long-term static load and subsequently subjected to different short-term static or dynamic loads, which is the only loading scenario that could realize significant benefit from this form of strength increase.

Strength Decrease Due to Repeated Impact Loads. An impact load of a magnitude greater than allowable short-term static loads will normally cause a temporary increase in excess pore pressure and consequent decrease in soil strength and anchor resistance to additional impact loads. This effect and the similar increases in excess pore pressures associated with cyclic loads of even lower magnitudes are discussed in a subsequent section entitled Effect of Load History.

Earthquakes

Large earthquakes cause shear stresses in the soil profile which. for granular soils, tend to be critically large at soil depths where embedment anchors are typically keyed. The locations worldwide where larger (magnitudes of 5 and greater on the Richter scale) earthquakes typically occur are well documented; Figure 7 illustrates where such earthquakes have occurred in the past. Anchor sites in these regions (within 100 miles of epicenters indicated in Figure 7 or within a similar distance of a band connecting and including the obvious zones of high major seismic activity indicated in Figure 7) should consider the possible effects of earthquake loading on anchor capacity; this is described in the section entitled Design Procedures for Cyclic/Repeated Loading Conditions.

QUASI-STATIC HOLDING CAPACITY

Definitions and Line Angle Effect

The quasi-static holding capacity, as defined earlier, is the load at which an embedment anchor will fail when this load is applied in a smoothly increasing fashion over a period of 1 to 15 minutes. Allowable quasi-static loads equal this holding capacity divided by a suitable F_s, as discussed later in this section. Water depth has no measurable effect on holding capacity.

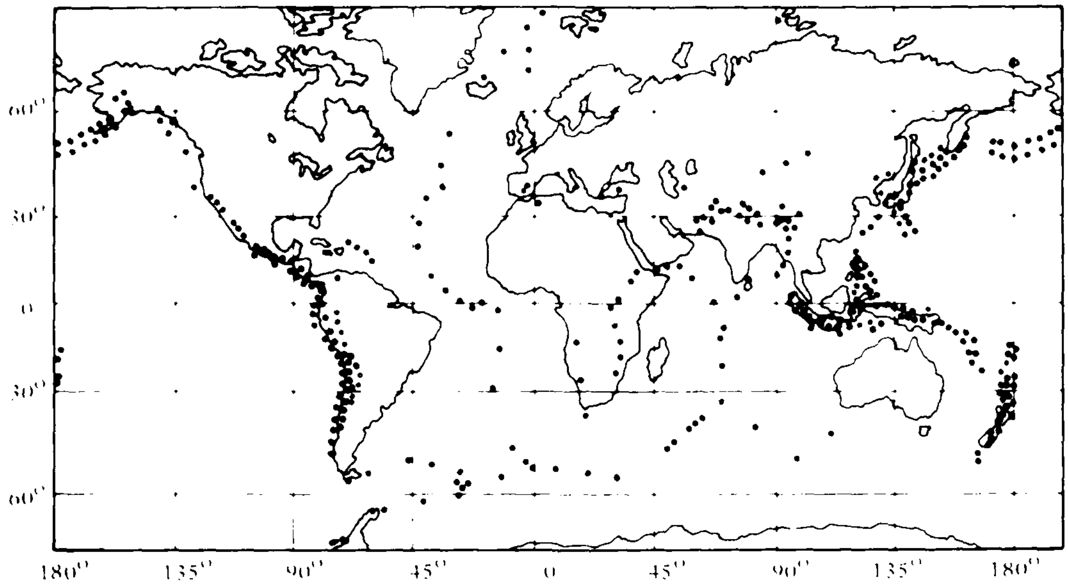


Figure 7. Worldwide distribution of past major earthquakes (from Wilson, 1969).

The calculated holding capacity is for vertical loading. For loads applied at other orientations, load capacity is assumed to be equal to that for vertical loading. If loading is at a 45-degree or larger angle relative to the vertical and the lateral direction of this loading can vary significantly (more than 90 degrees in azimuth, as in a single anchor single-point mooring where the lateral loading direction might vary 180 degrees with each tide cycle reversal), an increase in F_s is recommended. The normally applied F_s should be increased between 0% and 250% as the possible inclination of the application varies from 45 to 75 degrees from the vertical, and the increase remains at 250% for inclinations from 75 to 90 degrees from the vertical.

The capacity of an embedment anchor is heavily influenced by its depth of embedment in two major ways. For the first, in almost all instances applicable to these guidelines, soil strength increases with depth - thus, with all other things considered, the deeper the penetration and subsequent keying depth, the larger the anchor's holding capacity under all loading conditions. The second way is illustrated in Figure 8, where different shearing zones (the configurations of which control holding capacity) are indicated at different soil depths. A "deep anchor" shearing zone results in significantly larger holding capacities. To achieve a deep anchor, the D/B ratio (as illustrated in Figure 8) must be greater than 7 (slightly less for cohesive and for weaker or looser soils).

For anchor-holding capacity prediction, anchor embedment must be predicted (and later verified) or measured (in the case of an already installed and keyed anchor). The recommended procedures for obtaining the necessary information with these two approaches are summarized in the following subsections.

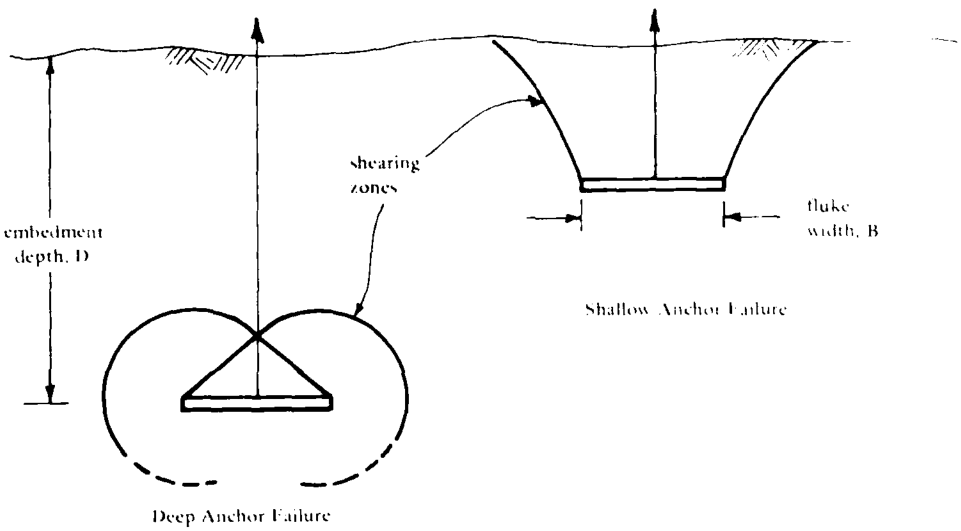


Figure 8. Behavior mechanisms for deep and shallow anchors (after Beard, 1980).

Anchor Penetration Prediction

The following procedure recommended for predicting anchor fluke penetration is based on an analytical model developed by True (1975) and is a slightly modified, and greatly simplified, version of that described in the publication by Taylor et al. (1975). The nature of the problem precludes use of a closed-form solution; thus, an incremental technique is used wherein the soil profile to the expected depth of fluke penetration (see Table 1) is divided into at least 20 increments of equal thickness Δz. The fluke velocity at the base of each soil increment, v_{i+1}, is calculated using the velocities for the two soil increments above, v_{i-1} and v_i, together with applicable characteristics of the anchor system and soil properties for that soil increment*. The following equation is used:

$$v_{i+1} = v_{i-1} + \frac{47\ \Delta z}{W_T\ v_i}\left[0.85\ W_T - \frac{v_i^2\ A_F}{92}(64 + \gamma_{bi}) - s_{ui}\left(9\ A_F + 0.66\ \frac{A_{si}}{S_{Ti}}\right)\left(\frac{4}{1 + \frac{1}{\sqrt{\frac{4\ v_i}{s_{ui}\ T} + 0.06}}}\right)\right] \qquad (1)$$

where

v = velocity of the anchor fluke (fps),

Δz = soil increment thickness (ft), can be arbitrarily set but suggest using $\Delta z \leqq 1/20$ x (expected total penetration from Table 1)

W_T = total in-air weight of the anchor fluke and piston (lb), see Table 1 or anchor system *specifications*

A_F = frontal area of anchor fluke (ft^2), see Table 1 or anchor system specifications

γ_b = buoyant unit weight of soil (pcf)

A_{si} = side area of anchor fluke and piston in contact with soil (ft^2). A_{si} equals A_s (see Table 1 or anchor specifications), except for the first several increments where the fluke and piston have not yet fully entered the soil. In those cases where $z_i < \ell_T$, approximate A_{si} with,

$$A_{si} = \frac{z_i}{\ell_T} A_s \qquad (2)$$

ℓ_T = length of fluke and piston (ft)

z_i = soil depth (ft) being considered for increment i,

$$z_i = i\ \Delta z \qquad (3)$$

---

*Increment number i where i starts at 1 and is incremented until $v_{i+1} \leq 0$, about 20 increments.

S_{Ti} = sensitivity (dimensionless), assume equal to 1.0 for granular soils

s_{ui} = undrained sediment shear strength (pcf) at a soil depth of z_i. For sands, the controlling dynamic undrained shear strength is largely a function of critical confining stress and is relatively constant with soil depth. See Table 3 for typical values as a function of density. Typical seafloor sands range from loose to dense.

T = fluke thickness (ft), can approximate as A_F/B

Subscript i = value of the property for the increment i being evaluated; in other words, at a depth of penetration or soil depth z_i.

Table 3. Effective Undrained Shear Strength of Granular Soils During Dynamic Fluke Penetration (Based Partially on Data From Castro and Poulos, 1976, and Conversations with True)

Density	Relative Density (%)	Effective Dynamic s_u (psf)
Very Loose	0 - 15	2,200
Loose	15 - 35	3,200
Medium	35 - 65	6,500
Dense	65 - 85	11,000
Very Dense	85 - 100	19,400

To initiate the iterative procedure, set i = 1 and solve Equation 1, assuming $v_1 = v_o$ where v_o is the initial fluke velocity (Table 1). Using the resulting calculated value of v_2, re-evaluate v_1 using the following,

$$v_1 = \frac{v_0 + v_2}{2} \tag{4}$$

Recalculate Equation 1 for i = 1, using this new value of v_1. Then continue the iteration for i = 2, 3, ... until $v_{i+1} \leq 0$. At that point the expected depth of maximum fluke tip penetration for the anchor and site conditions specified is determined from,

$$D_{pk} = i \, \Delta z \tag{5}$$

Anchor Keying

As illustrated in Figure 3 and mentioned earlier, once embedded, the anchor fluke must be pulled upward to key it. This keying distance for a properly designed anchor fluke is 1 to 1-1/2 fluke lengths in sand, and 1-1/2 to 2 fluke lengths in cohesive soils. Figure 9 illustrates typical behavior during keying in a cohesive soil. To determine the final keyed depth of embedment, D, it is suggested that the larger typical keying distance be used. Thus, depth of embedment can be estimated from the following,

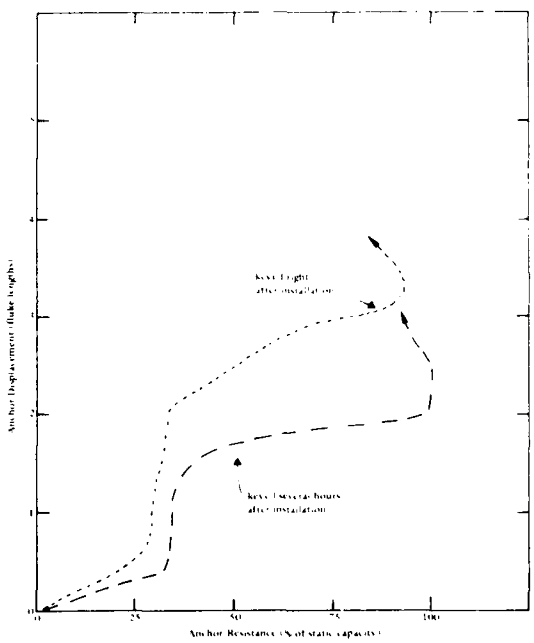


Figure 9. Example of data obtained during anchor keying (after Rocker, 1977).

For clays or mud:

$$D = D_{pk} - 2\,L \tag{6}$$

where L = fluke length (ft)

For sand:

$$D = D_{pk} - 1.5\ L \tag{7}$$

To key the anchor fluke properly, a significant vertical force must be applied to the anchor cable. One-third to one-half of the ultimate quasi-static holding capacity of the anchor is the suggested force magnitude (see the report section which follows for determining this capacity). The magnitude may be varied if requirements for proof testing are different or if the expected in-service loading levels are significantly different. The anchor fluke need not be keyed fully initially for it to function properly; large in-service loads subsequently applied will fully key the fluke.

For anchors in cohesive soils, it is recommended that the time between firing the fluke into the sediment and pulling on the anchor for keying be greater than 1 hour - longer, if possible. This results in shorter keying distances (Rocker, 1977) and increased anchor holding capacity. The data in Figure 9 illustrate this behavior pattern. It is realized that from the standpoint of operating procedures this may not be the most straightforward way of sequencing steps and may, in fact, not be practical in some situations. However, a delay between the installation and the keying of the fluke of several hours (a 1-day delay is preferable) typically results in a 10% to 20% increase in holding capacity. The design procedures in this report assume at least this 1-hour delay before keying in cohesive soils. If this is not operationally feasible, the calculated quasi-static anchor capacity should be reduced by 20%.

Anchor Verification

Verification of both keyed anchor depth and holding capacity is highly recommended. Holding capacity is usually verified by a proof test wherein the anchor is loaded quasi-statically to its design or allowable level F_{all} either vertically or in the orientation of its operational loading. Verification of keyed anchor depth is also important, even

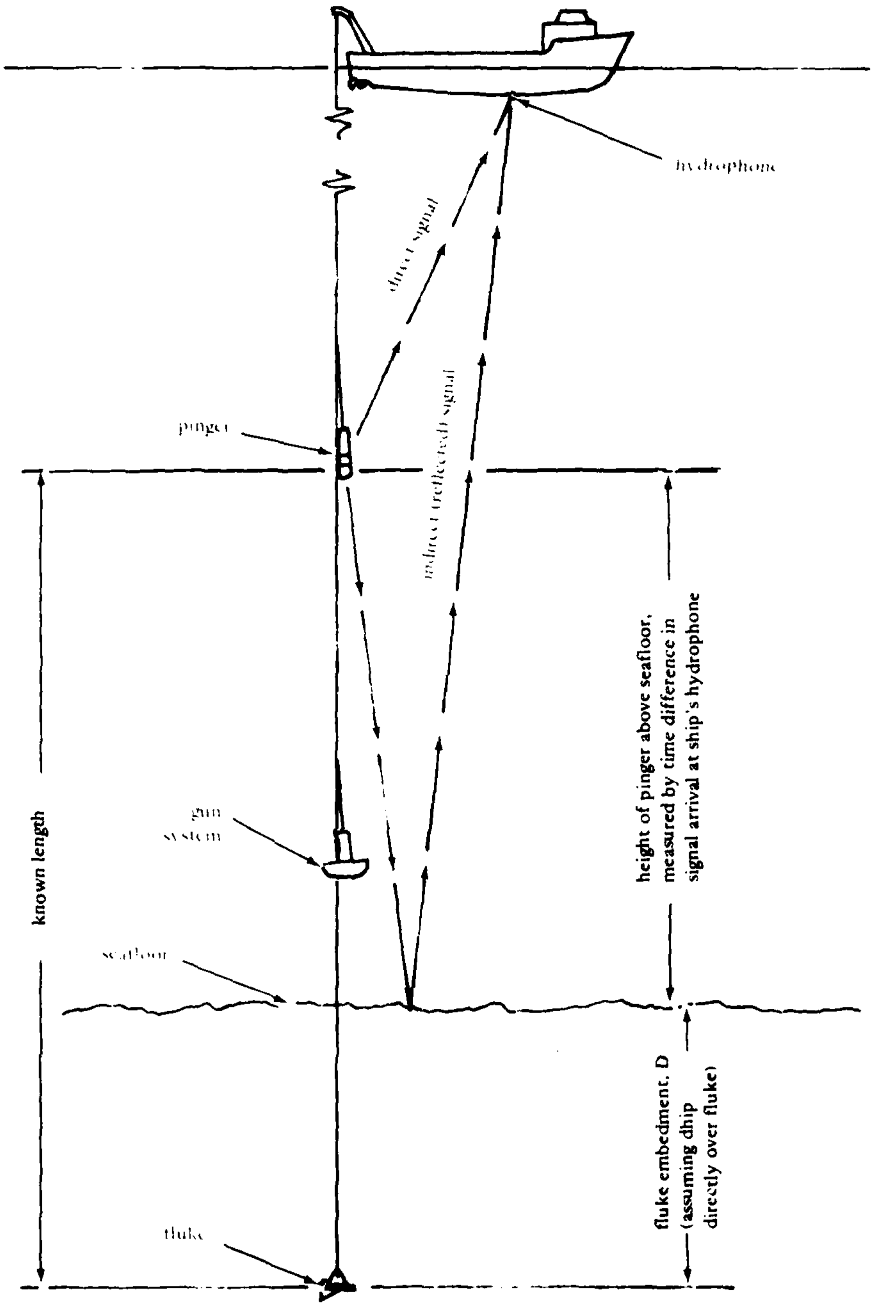


Figure 10. Schematic of anchor keying verification technique using an acoustic pinger (from Valent, 1978).

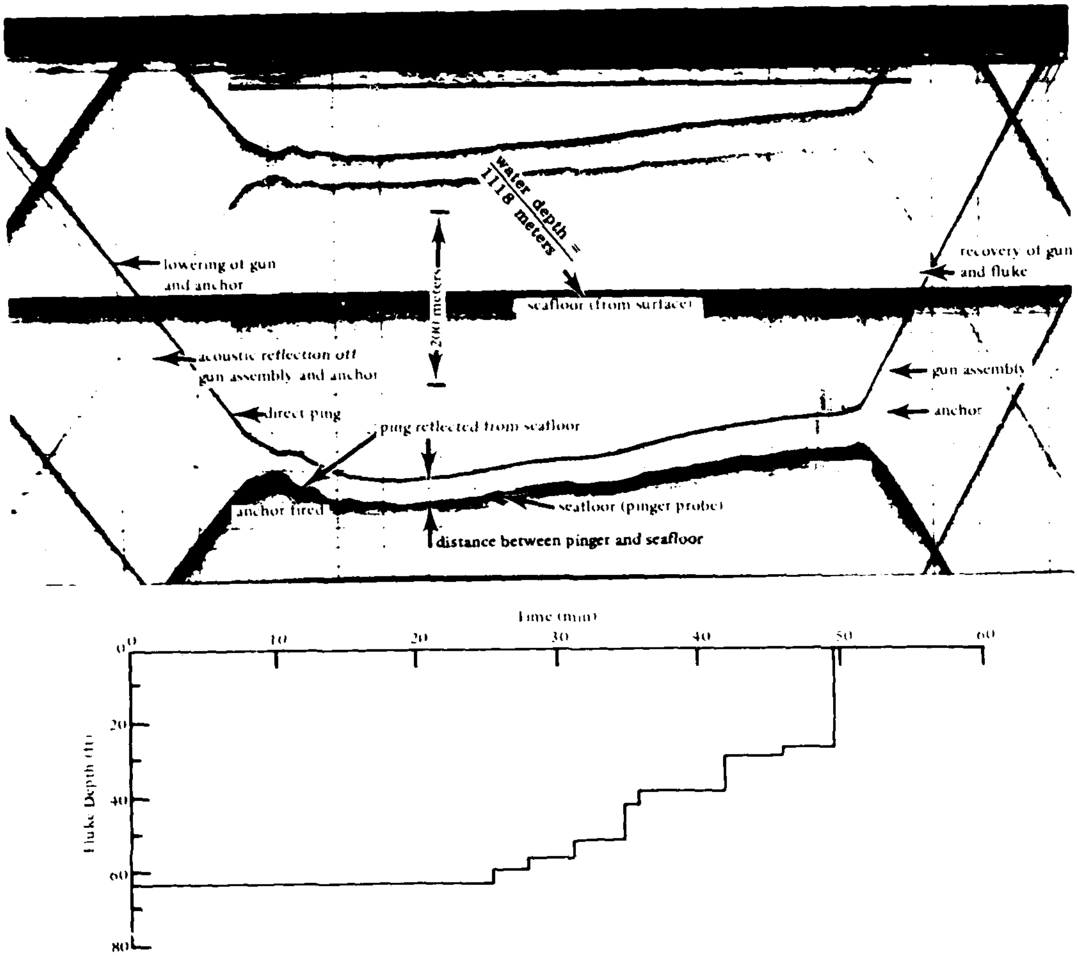


Figure 11. Example data showing anchor keying and slow load test to failure in deep water (after Valent, 1978).

when a quasi-static loading test is conducted, because the performance under dynamic conditions is heavily influenced by depth of embedment and is not verified by simply successfully loading the anchor to its quasi-static design level. A true static verification requires monitoring anchor displacement under loading; the means for accomplishing this, typically, also measures depth of embedment.

In shallow water, divers can attach simple systems (marks or reference points) for measuring movement, and depth of embedment can be determined by measuring the length of downhaul cable extending out of the seafloor. In deeper water, the acoustic procedure illustrated in Figure 10 is recommended. Data from Malloy and Valent (1978) resulting from a typical use of this deep water procedure are illustrated in Figure 11 for a test in 3,000 feet of water where an anchor was loaded to failure in 49 minutes. The widening gap between the "Direct Ping" and the "Ping Reflected from Surface" in the upper half of Figure 11 indicates the progressive pullout of the fluke in this test to failure. The lower half of Figure 11 presents a simplified view of fluke depth versus time and includes corrections for wire angle at the seafloor. The peak load on the fluke at the apparent time of each movement of the fluke is indicated. The record of raw data does not appear to show the distinct upward movements of the fluke indicated. This is because the cable is going relatively slack between peak loads which gives a smooth appearance to that record. Detailed examination and analysis of the raw data (including both displacement and load level records) is required in order to properly determine actual behavior in a longer duration test to failure such as this one. The records obtained from a typical anchor setting/keying and proof testing operation are much shorter and simpler to interpret. More detailed information on this procedure can be found in Malloy and Valent (1978) or in Valent (1978). The latter report discusses several case histories, including problems encountered with interpretation and the means for overcoming them.

Holding Capacity Determination

The ideal method for determining holding capacity of an embedment anchor is to load it to failure. This use of test anchors as a direct analogy of test piles may be a practical approach in an area of uniform soil conditions where a number of identical anchors are to be installed. Subsequent service anchors should still be proof-loaded to their design operating levels, and their individual records of load versus fluke movement should be checked for similarity to those for the test anchors.

For the typical situation where test anchors are not possible, anchor capacity can be predicted using the equations below. The soil at the site must be classified as a granular soil (sand) or cohesive soil (mud or clay) as outlined earlier.

<u>Cohesive Soils</u>. The short-term quasi-static holding capacity of an embedment anchor in cohesive soils can be calculated using the following,

$$F_T = \bar{N}_c \, A \, f \, s_u \, (0.84 + 0.16 \, B/L) \qquad (8)$$

where F_T = short-term holding capacity (<u>not</u> allowable load) (lb)

$\bar{N}_c$ = holding capacity factor, equals 15 except for some shallower burial anchors defined in Figure 12a. These values are based on full suction, which is appropriate only for the short-term loading conditions addressed in this report. For loads of longer duration for which suction cannot be relied on, a reduction of the $\bar{N}_c$ factors from Figure 12a by 40% is necessary. (dimensionless)

A = gross bearing area of fluke after keying (ft^2)

f = disturbance factor, assumed = 0.7 for cohesive soils addressed by this procedure (dimensionless)

s_u = short-term undrained shear strength of the soil at a soil depth of D - B/2 (i.e., at a critical soil depth just above the keyed anchor fluke) (psf)

B = anchor fluke gross average width (ft)

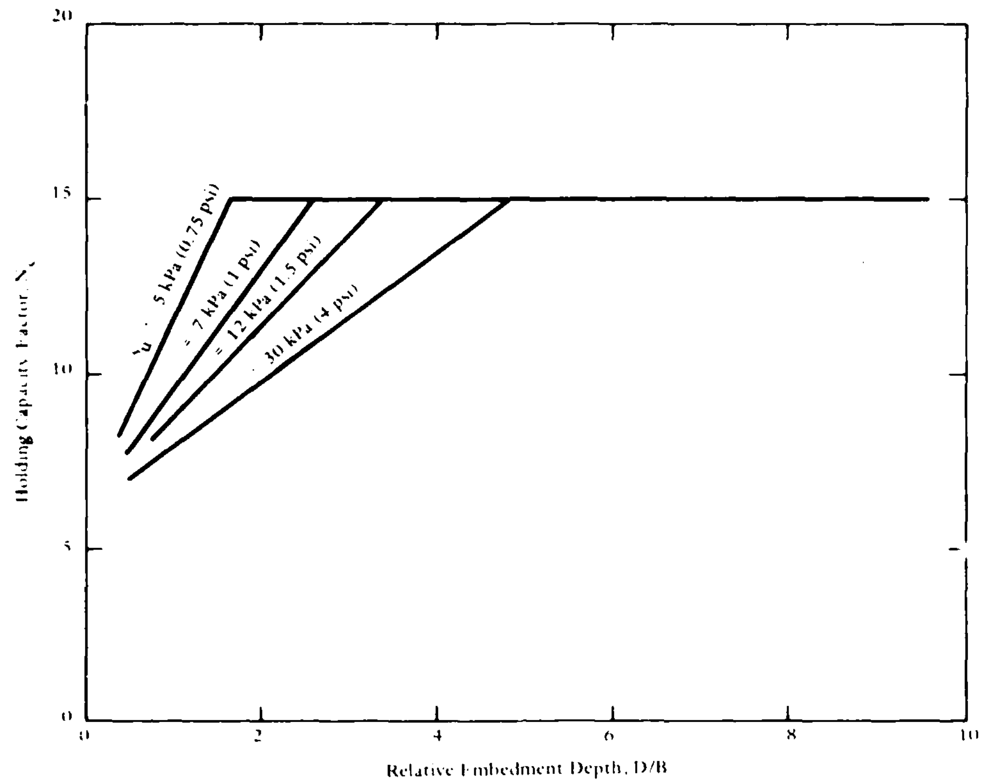
L = anchor fluke gross average length (ft)

<u>Granular Soils</u>. The quasi-static holding capacity of an embedment anchor in granular soils can be calculated from the following (which assumes no suction and fully drained conditions),

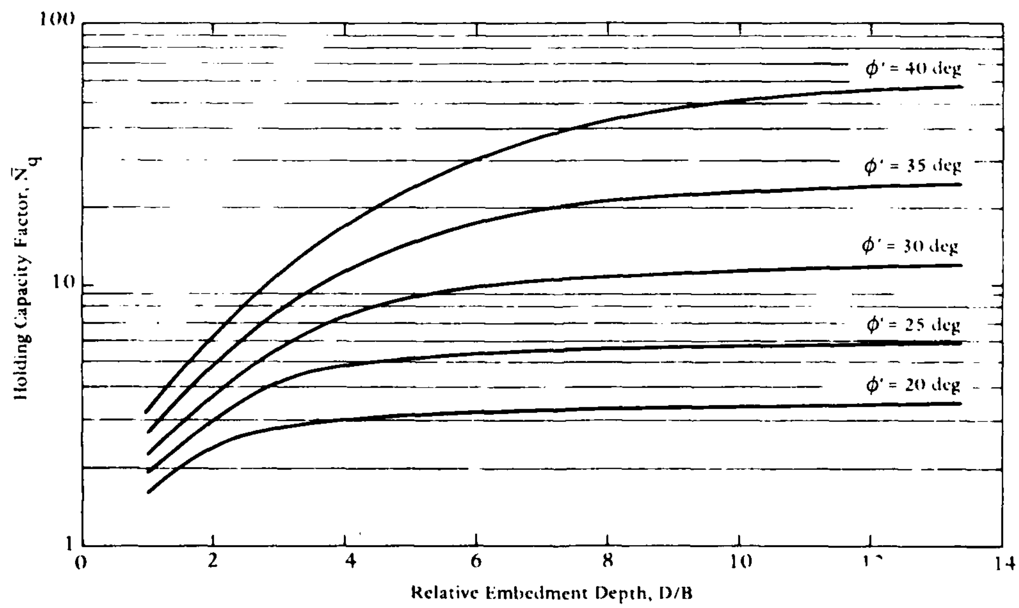
$$F_T = \bar{N}_q \, A \, \gamma_b \, D \, [0.84 + 0.16 \, B/L] \qquad (9)$$

where $\bar{N}_q$ = holding capacity factor, which is a function of the soils friction angle, ϕ', as defined in the following table (dimensionless) and in more detail in Figure 12b

Friction Angle, ϕ'	$\bar{N}_q$	Minimum D/B
20	2.8	3
25	5.5	3.5
30	8	5
35	20	7
40	40	8



(a) Short-term holding capacity factors for cohesive soils (clays and muds).



(b) Holding capacity factors for cohesionless soils (sands).

Figure 12. Evaluation of anchor holding capacity factors (from Beard, 1980).

γ_b = submerged unit weight of the soil (pcf)
D = soil depth of fluke after keying (ft)

Other variables are defined as discussed in the paragraph entitled Cohesive Soils.

Allowable Loads - Factor of Safety

The allowable quasi-static load on an embedment anchor F_{all} equals the holding capacity of the anchor, as determined by the methods discussed earlier, divided by an appropriate factor of safety F_s.

$$F_{all} = F_T/F_s \tag{10}$$

Appropriate values for F_s for typical applications of embedment anchors range from 1.5 to 3.0. Low values are appropriate where there is high confidence in determinations of loading conditions, soil properties, and expected anchor behavior (such as those instances where proof loads are applied), and where the consequences of anchor failure are not severe. Large values of F_s are needed where confidence in determined values is lower and where the consequence of failure is more severe. In cases where site conditions cannot be measured and, as a result, must be assumed, a value of F_s greater than 3.0 should be considered unless proof testing can be conducted. For most past embedment anchor installations where site properties have been properly determined in advance and anchor keying/setting and fluke depth verification have occurred, values of F_s between 1.5 and 2.5 have been used (the exact valuation being dependent upon the consequence of anchor failure, should it occur). A value of at least 2.0 is recommended for most normal applications.

DYNAMIC LOADING CONDITIONS

Dynamic loads are defined here as those which are applied rapidly, with a duration of less than 1 minute and often in a repetitive or cyclic fashion. All loads discussed here are those applied to the anchor fluke in the seafloor. These loads may be quite different from the loads applied elsewhere in a system, such as to a surface buoy in a mooring system. Dynamic loading conditions are probably the most common and most severe types of loading on seafloor embedment-type anchors. Dynamic load conditions have been divided into two categories: impact loading (basically single events) and repetitive or cyclic loading. Of course, an anchor can be subjected to both types of loading. The differences between impact and cyclic loads, along with several definitions related to dynamic loading conditions, are illustrated in Figure 13. These two basic types of loading, along with earthquake loading of the soil mass (a specialized case of cyclic loading), are further defined and examples given in the sections which follow.

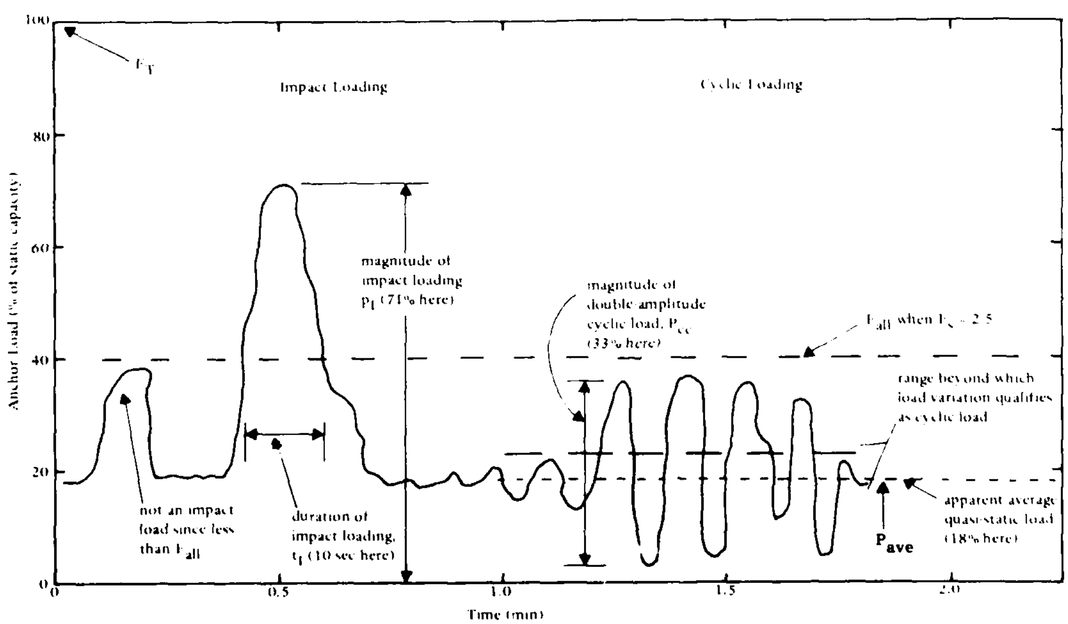


Figure 13. Example of dynamic time-load histories, illustrating definitions and parameters.

Impact Loads

Sources. Impact loads can result from several sources, including the following:

(1) Initial impact loading as a ship is unintentionally driven into a mooring (or when a ship is attempting to "break" a mooring for departure)

(2) Forces during installation or construction operations, such as bottom-up release of an array, momentary tensioning of a system to straighten out or realign cables or components, or unintentional tensioning of a leg by the installing vessel during construction of a multi-legged deep water mooring system or array

(3) Blast effects due to ordnance explosions or similar events

Definition/Quantification (refer to Figure 13). Impact loads requiring consideration in these design procedures are defined as those which are greater in magnitude than the allowable static load on an anchor (F_{all}), less than 1 minute in duration (if longer, they would qualify as an excessively large quasi-static load), and not rhythmic in nature or repeated more than five times during the characteristic excess pore pressure dissipation time, t_{cd}. (See the subsequent section entitled Effect of Load History for precise definition and evaluation of this parameter.) If impact loads are repeated more frequently, they qualify as a cyclic load. An impact load is characterized for the purpose of analysis in terms of the magnitude of the impact load, P_I, (usually expressed as a percentage of the anchor's quasi-static holding capacity) and its duration, t_I, above the allowable quasi-static load level, F_{all}.

Cyclic Loads

Sources. Cyclic loads typically result from cable strumming, surface-wave-induced forces, and earthquake loading.* Cable strumming is induced in relatively taut cables by a passing current. This is a higher frequency phenomenon (typically 5 to 20 Hertz for practical situations) with load magnitudes sufficiently low that they can be ignored when using the simplified approach of the guidelines in this report. The effects of strumming on fatigue of cable and mechanical components immediately above the seafloor may not be ignorable, however. Surface and subsurface buoys and ships/platforms riding in a mooring can all induce significant cyclic loads in an anchor, usually at the frequency of the waves - typically 0.05 to 0.15 Hertz for significant loading.

Definition/Quantification (refer to Figure 13). A cyclic load must have a double amplitude greater than 5% of quasi-static anchor capacity for the loading to be considered cyclic from a design standpoint. Smaller cyclic loads are difficult to measure or predict and can be ignored in the design.

Cyclic loads are characterized in terms of the average quasi-static load on which the double-amplitude cyclic load is superimposed. These two magnitudes should be expressed in terms of their percentage of the quasi-static anchor holding capacity. The other parameter needed is the number of cycles of loading which occur. In fact, two different counts of number of loading cycles are needed - N_T and N_D. N_T is the total number of cycles to which an anchor is subjected during its lifetime; this number is used in evaluating the potential for cyclic creep. N_D is the number of cycles that occur in a shorter period of time during which dissipation of excess pore pressure is not large (e.g., for the case of an anchor in clayey silt, this may be the duration of a major storm). This period is taken as t_{cd}, which is defined

*In the earthquake loading case, the entire soil mass (rather than the anchor fluke and immediately adjacent soil) is loaded in shear. This is a special case which is treated in the section entitled Earthquake Loading.

in the section entitled Effect of Load History. This number is used to evaluate the potential for a strength loss or liquefaction failure. Cycles occurring outside this time period are not counted because there is sufficient time for dissipation of excess pore pressure (see the section entitled Effect of Load History for further explanation).

Typically, many periods of significant cyclic loading (perhaps individually as long as t_{cd}) occur during the life of an anchor. If the magnitude of cyclic loading is relatively constant over the life of the anchor, the most critical period of cyclic loading, and, thus, the one to be analyzed, is the first one. If larger magnitudes of cyclic loading are expected at later times, several periods – earlier periods of smaller magnitude cyclic loading and the periods of subsequently larger cyclic loading - should be checked to see which is the most critical. The effects of load history should be considered in this latter analysis.

The procedure outlined above is quite straightforward when the cyclic loads are of relatively uniform magnitude (double-amplitude cycles P_{cc}, the same within ±10% relative magnitudes), or when a major portion of the cyclic loads (e.g., one-third) are relatively uniform and are significantly larger (by 50% relative magnitudes) than the rest of the cyclic loads. In this latter case, the smaller two-thirds of the waves can be ignored. For other cases where the spectral distribution of cyclic load magnitudes exhibits a tail of extreme values, such as that illustrated by the wave height spectrum for a 100-year storm in the North Sea shown in Figure 14a, a different approach is required.

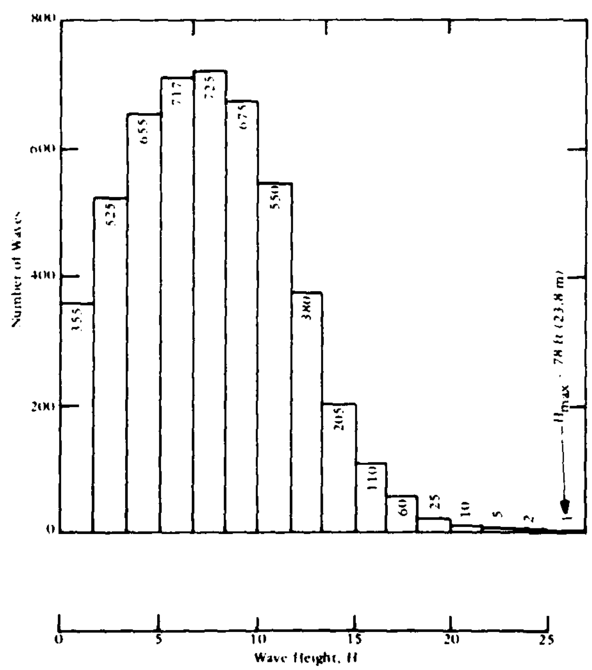


Figure 14 (a) Distribution of wave heights in an example 100-year storm (from Lee and Focht, 1975. Used by permission of ASCE).

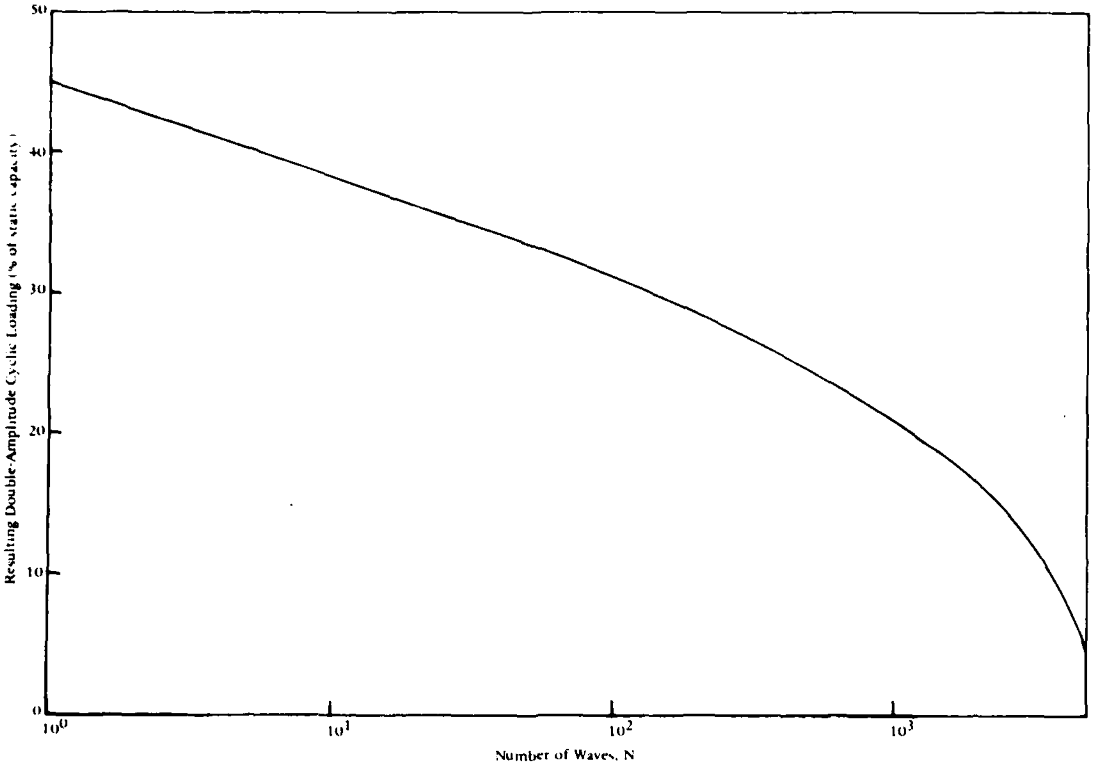
Table 4. Example Critical Wave Loading Analysis of an Anchor System (Based partially on data from Lee and Focht, 1975)

Wave Height, H_i (ft)	Number of Waves, N_i	Cumulative Number of Waves	Resulting Cyclic Anchor Load, P_{ui} (% of static capacity)	Analysis for Equivalent Uniform Wave Loading*			
				Load Categories, P_{ui}	Number of Cycles, N_i	N_{if}	ΔN_{eq}
0-5	355	5,000	2	Since these loads are less than 50% of maximum cyclic load, these two-thirds of load cycles can be ignored.			
5-10	525	4,645	5				
10-15	655	4,120	8				
15-20	717	3,465	11				
20-25	725	2,748	13				
25-30	675	2,023	16				
30-35	550	1,348	19	21	930	300,000	6
35-40	380	798	22				
40-45	205	418	25	27	315	25,000	25
45-50	110	213	28				
50-55	60	103	31	33	85	2,000	85
55-60	25	43	34				
60-65	10	18	36	38	15	500	60
65-70	5	8	39				
70-75	2	3	42	44	3	200	30
75-80	1	1	45				
Total	5,000					N_{eq} =	206

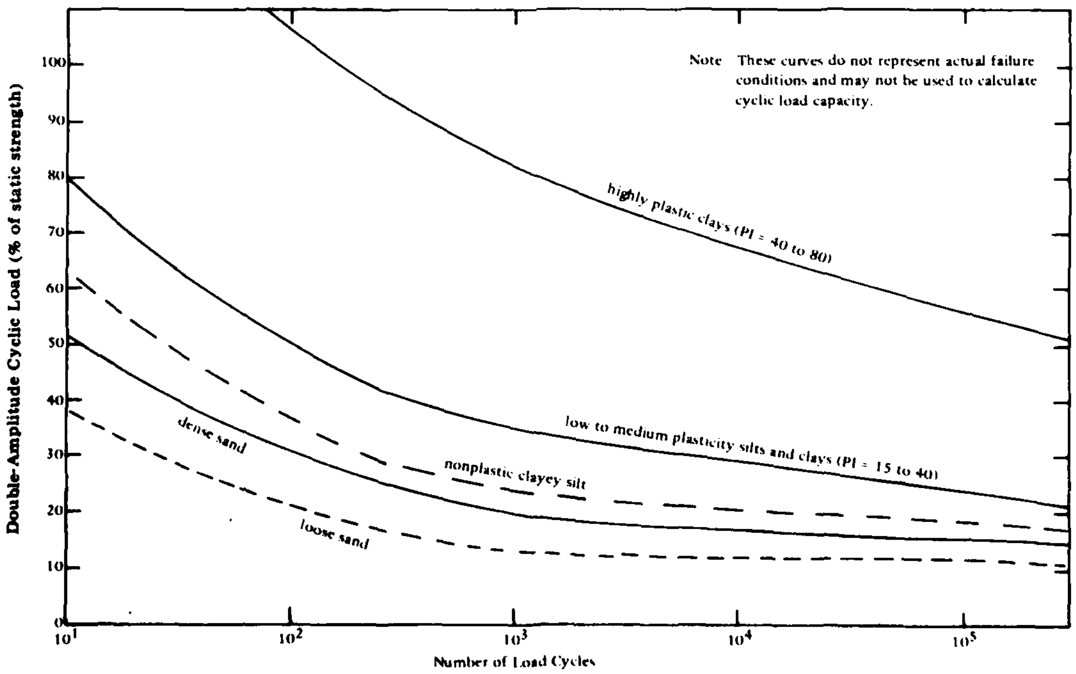
*P_{ceq} taken as 33%; from Figure 13c, N_{ref} is 2,000 using the contour for clayey silt.

An example of the loads resulting from the wave height spectrum presented in Figure 14a are summarized in Table 4 and in Figure 14b. This loading spectrum can be divided into segments of relatively uniform magnitude. This procedure is illustrated in Table 4 where the spectrum of Figure 14a is broken into sections, within each of which the cyclic loads can be described as being of "relatively uniform magnitude," according to the definition established earlier. These individual segments of the cyclic loading spectrum can then be summarized in terms of the average double amplitude cyclic load and the number of loading cycles within each. This method of summarization into a number of equivalent uniform loading cycles is based on the procedure developed by Lee and Focht (1975).

The significant cyclic loads expected during a time t_{cd} are summarized as illustrated in Table 4. If there are several extreme events (fewer than five), these can be treated as either cyclic or impact loads, assuming they are large enough to be considered impact loads (i.e., $P_I \geq F_{all}$). For illustration purposes, they are handled as cyclic loads here. For a broad spectrum of load magnitude, such as that illustrated in Table 4, the smaller one-half to two-thirds can be ignored. The larger loading cycles are grouped into sections/categories of relatively uniform magnitude (similar within ±5% to 10%). The effect of these nonuniform cyclic loads is assessed by converting to an equivalent number of uniform cyclic loads using an approach that might be termed equivalent damage theory. The curves in Figure 14c indicate, for several categories of soil type, combinations of cyclic load level and number of cycles that cause failure (i.e., any of the combinations making up the curve for a given soil cause equivalent damage – failure). The values of expected cyclic load P_{cc} are examined, and an equivalent uniform value P_{ceq} is selected such that P_{ceq} equals 65% to 85% of the maximum value of P_{cc} expected. The lower percentage is used when there is a tail of extreme high values of P_{cc} in the spectrum being considered. The higher percentage is used when the spectrum being considered is more uniform and there are no extreme high values of P_{cc}



14 (b) Resulting cyclic loading of anchor by a 100-year storm.



14 (c) Significant damage contours as a function of soil type.

or those that did exist are treated as impact loads rather than as cyclic loads. The number of equivalent cycles, N_{eq}, of uniform magnitude P_{ceq} is then determined from the following

$$N_{eq} = \sum_{i=1}^{N_{ru}} \Delta N_{eq} \qquad (11)$$

where N_{ru} = number of group/categories of relatively uniform cyclic loading

$$\Delta N_{eq} = \frac{N_i}{N_{if}} N_{ref} \qquad (12)$$

where N_i = number of loading cycles in the actual load spectrum of relatively uniform magnitude, P_{cci}

N_{ref} = number of cycles at magnitude P_{ceq} to cause significant damage according to Figure 14c

N_{if} = number of cycles of magnitude P_{cci} required to cause significant damage according to Figure 14c

While the curves in Figure 14c are referred to as significant damage contours, these are not truly criteria against which the sufficiency of the anchor design can be assessed. That assessment is made in the section entitled Design Procedures for Cyclic/Repeated Loading Conditions.

It should be noted that the example used and illustrated in Figure 14 is unusual in its completeness. For such cases, it is also possible to simply compare the resulting soil loading spectrum, Figure 14b for example, to the appropriate criteria in the subsequent section entitled Design Procedures for Cyclic/Repeated Loading Conditions.

Predicting forces on an anchor resulting from wave forces on a buoy or moored platform is a more complex and difficult procedure. Such forces are often predicted strictly in terms of maximum quasi-static forces, a procedure which is not satisfactory since cyclic loads of lower amplitude are often more damaging to anchor stability than larger quasi-static loads or even larger impact loads. Methods for predicting static and dynamic forces on moorings, including the forces at the anchors, are currently being developed by NCEL. Palo and Webster (1980) summarizes current capabilities and references related work. The shape of the cyclic or impact loads (rapid rise time and slower decay, or sawtooth-, rectangular-, or sinusoidal-shaped cyclic loading) is not of concern in these guidelines since the procedures and parameters used have been defined in ways which account for the few significant differences caused by the shapes of dynamic loading histories.

Earthquake Loading

Earthquakes are a cyclic loading (usually at a frequency of about 2 Hertz and with 10 to 30 significant loading cycles, depending upon the magnitude of the earthquake), which differ from the preceding category in that the cyclic loading is induced in the entire soil mass (rather than into the anchor from above) by the earthquake energy radiating up and out from the epicenter or causative fault. The geographical locations of past major earthquakes and, thus, likely future ones are illustrated in Figure 7. The maximum accelerations induced in the soil mass by major earthquakes are a function of the earthquake magnitude and the distance of the site from the earthquake epicenter or, more precisely, from the causative fault. Predictions of these accelerations are summarized in Figure 15. Methods for

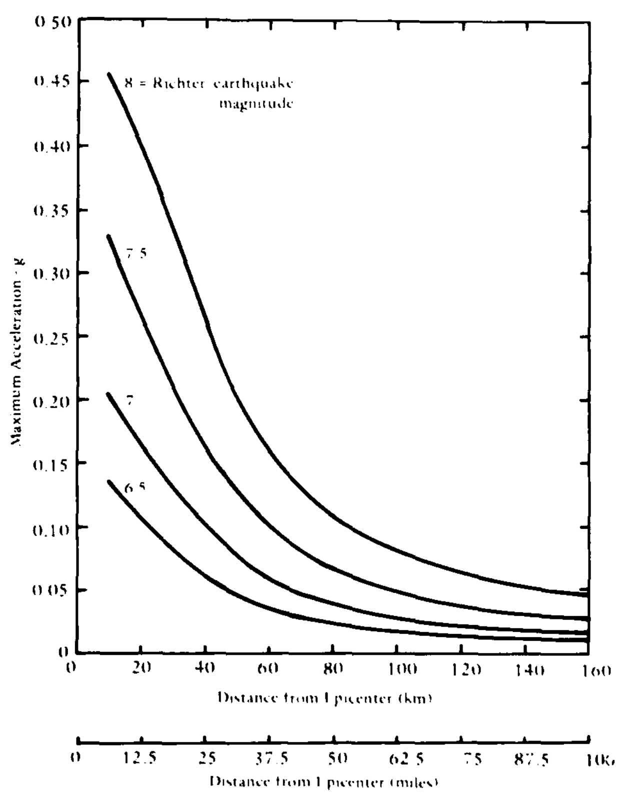


Figure 15. Maximum accelerations associated with earthquakes of various magnitudes (from Seed et al., 1969. Used by permission of ASCE).

predicting anchor stability under the earthquake loadings utilizing the above information are presented in a subsequent section entitled Design Procedures for Cyclic/Repeated Loading Conditions.

DESIGN PROCEDURES FOR DYNAMIC/IMPACT LOADING

The determination of anchor capacity under impact-type loading (loadings with duration of <1 minute) follows the same basic procedure as that used for determination of the short-term quasi-static capacity. The only differences are the value of undrained shear strength to be used, the use of an inertial factor in some cases, and the introduction of capacity reduction factors. The undrained shear strength under impact loading s_{uI} depends on the duration of the impact loading t_I. The definitions and characterizations of impact loads were presented in the preceding section. The approach used here utilizes modified versions of the equations used earlier for quasi-static short-term capacity determination - Equations 8 and 9 for cohesive and granular soils, respectively. The forms of the equations for impact loading are as follows.

For cohesive soils,

$$F_I = \bar{N}_c A f s_{uI} R_c R_I I_F (0.84 + 0.16 B/L) \tag{13}$$

For granular soils,

$$F_I = \bar{N}_{qI} A \gamma_b D R_c R_I I_F (0.84 + 0.16 B/L) \tag{14}$$

where F_I = the anchor capacity under impact loading (lb)

s_{uI} = the undrained shear strength mobilized under impact loading conditions where,

$$s_{uI} = I s_u \tag{15}$$

and I is the strength influence factor for impact loading

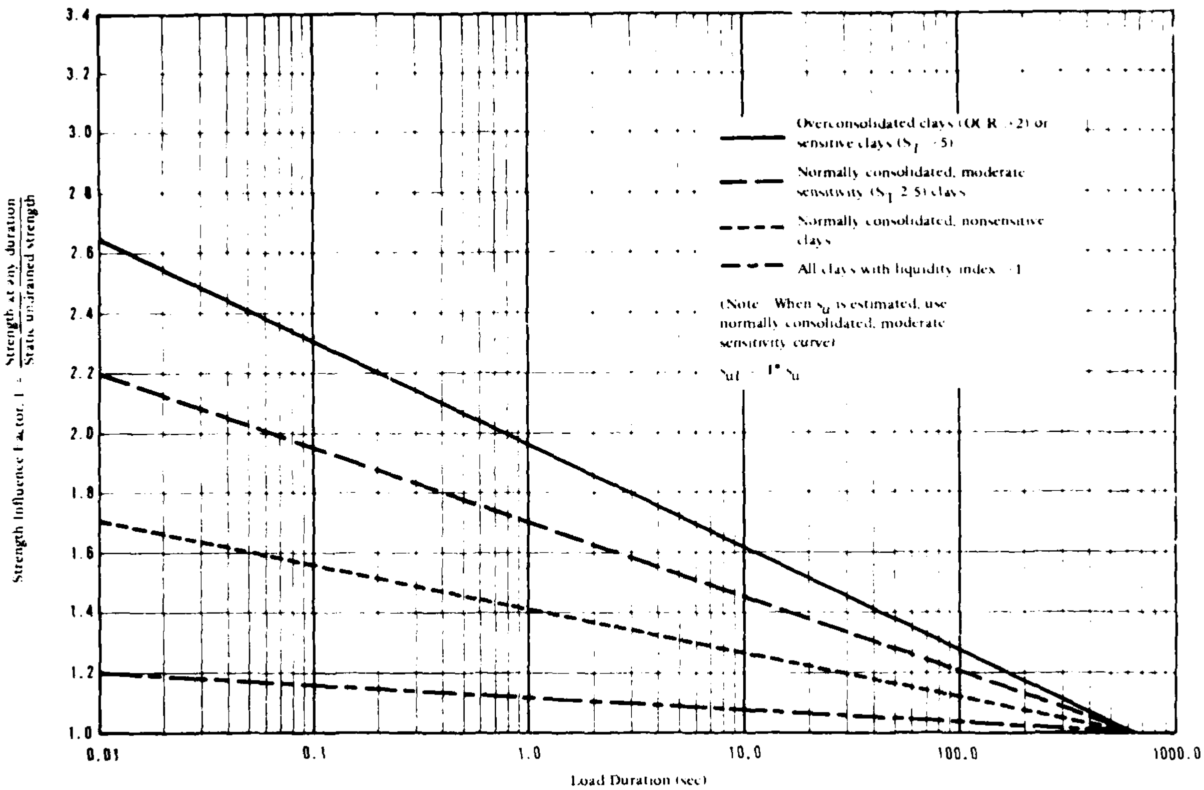


Figure 16. Impact duration influence factors for cohesive soils (from Douglas, 1978).

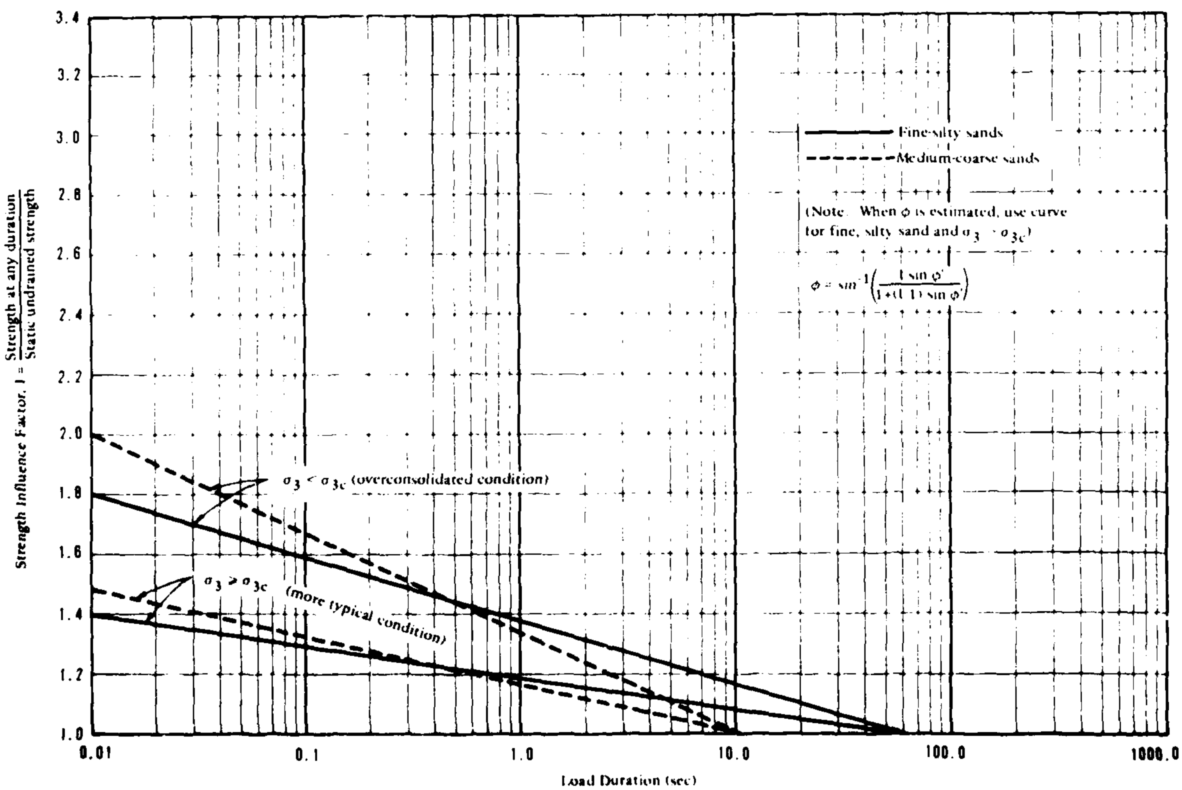


Figure 17. Impact duration influence factors for granular soils (from Douglas, 1978).

$\bar{N}_{qI}$ = the holding capacity factor applicable to impact loading conditions and $\bar{N}_{qI}$ is determined from a ϕ_I, the soils effective friction angle under impact loading, which is determined from the following,

$$\phi_I = \sin^{-1}\left[\frac{I \sin \phi'}{I + (I-1) \sin \phi'}\right] \tag{16}$$

R_c = the cyclic reduction factor based on cyclic loading history and determined in the subsequent section entitled Effect of Load History

R_I = the reduction factor for repeated impact loadings

I_F = the inertial factor applicable to very rapid loading conditions

All other parameters are as defined in the earlier section entitled Quasi-Static Holding Capacity.

The first step in design is to determine when the impact load will occur. *If the load will occur as the first event in the anchor history*, the reduction factor, R_c, for cyclic loads, will be equal to 1. If the impact loading is anticipated as occurring after a series of cyclic loads and within time period t_{cd} of their occurrence, then R_c shall be evaluated using the procedures described in the subsequent section entitled Effect of Load History.

The second step is to determine whether the impact load is to be a single event or a repeated event. If it is a single event, or an event not repeated within a time equal to 0.5 t_{cd}, then the reduction factor, R_I, should be set equal to 1. If the loading is to be repeated in times shorter than the above limit, then the capacity will be reduced by using an appropriate value of R_I as described in the subsequent section entitled Effect of Load History.

The third step is to determine the appropriate influence factor I to be applied to s_u or ϕ' for cohesive or granular soils, respectively. Using Figures 16 and 17, the soil strength, and the impact load characteristics that were determined using the procedures in an earlier section on Impact Loads, the designer can select the appropriate influence factor. The duration to be used is that estimated for the single impact of concern, t_I, as defined in the preceding section entitled Dynamic

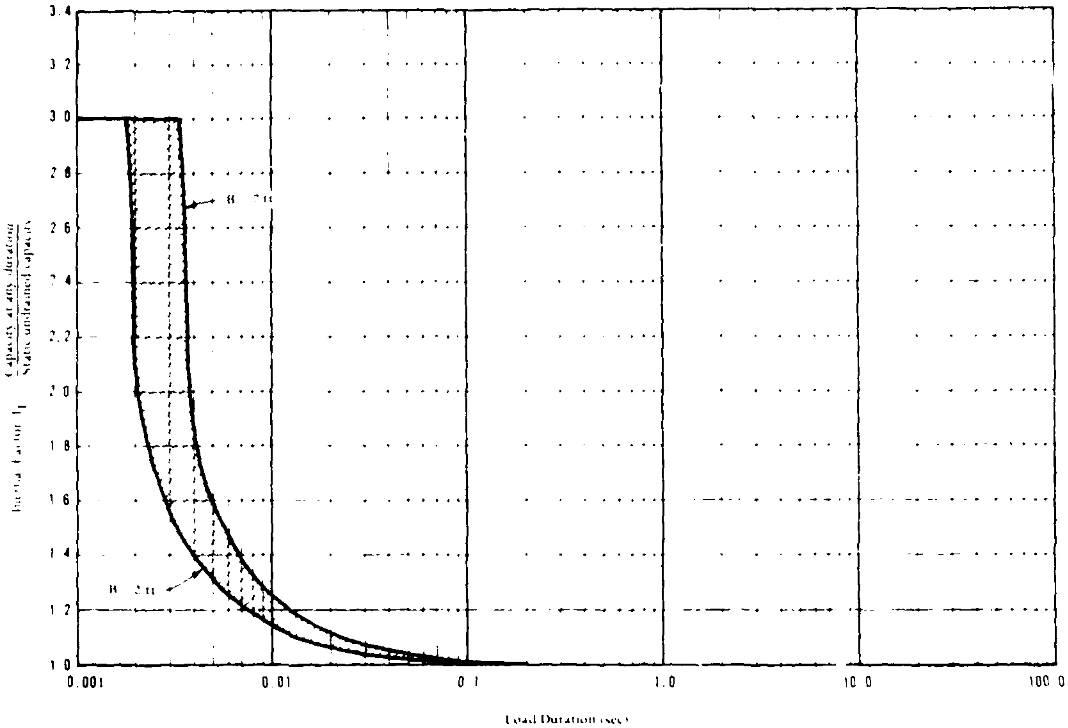


Figure 18. Inertial factors for NCEL-configured embedment anchors in sands and clays (from Douglas, 1978).

Loading Conditions and in Figure 13. For cohesive soils, the influence factor is determined from Figure 16 and is applied directly to the value of s_u as defined in Equation 15. The resultant value is termed the undrained shear strength under impact loading, s_{uI}. For granular soils, the influence factor I is determined from Figure 17 and is used to adjust the holding capacity factor by directly modifying the friction angle, ϕ', by use of Equation 16. The resultant value is the friction angle under impact loading, ϕ_I.

The fourth step is to check to see if the impact loading is expected to have a duration ≤0.02 second. If so, an inertial factor may be applied. The values, shown in Figure 18, are applied directly to the rated capacity as indicated in Equations 13 and 14 and are a function of both impact duration and, to a lesser extent, the size of the anchor fluke. Inertial factors I_F are used to account for any added capacity due to the inertial resistance of the anchor and attached mass of soil. This factor should only be used when load durations are known with certainty. Although the rise time determines the magnitude of potential inertial capacity increases, the duration of load will determine the actual capacity. In addition, because of the small mass ratio (mass of anchor divided by mass of attached soil), the system damping ratio is greater than critical. Hence, dynamic "overshoot" will not be of concern, and it will be duration, rather than rise time, that controls ultimate capacity. The evaluation of the inertial factor for impact loading I_F in Figure 18 assumes the geometric mass characteristics of the CEL family of embedment anchors. For other anchor systems with significantly different characteristics, Figure 18 may not be applicable. For such cases, the reader is referred to Douglas (1978).

The final step is to calculate the anchor capacity using the same procedure for short-term static determination, but basing s_{uI} or $\bar{N}_{qI}$ on ϕ_I (instead of s_u or $\bar{N}_q$ on ϕ') and using the equations modified for impact loading – Equations 13 and 14.

Each factor – R_c, R_I, I_F, and I – has been selected to yield conservative results, even for the worst condition. Additional conservatism results from the inability of the anchor-soil system to develop a continuous failure surface under very rapid loadings. For these reasons, no additional factors of safety are recommended. The factor of safety that is selected for use in short-term capacity determination is appropriate for application to the predicted impact capacity. Thus, the form of Equation 10 is applicable, and the allowable impact load F_{Iall} equals the anchor capacity under impact loading divided by the applicable factor of safety determined in the earlier subsection entitled, Allowable Loads – Factor of Safety. The appropriate equation is,

$$F_{Iall} = F_I/F_s \tag{17}$$

DESIGN PROCEDURES FOR CYCLIC/REPEATED LOADING CONDITIONS

Design procedures for cyclic/repeated loads are separated into three categories: (1) cyclic loading of the anchor which may lead to strength loss or a liquefaction-like relatively sudden anchor instability and failure; (2) cyclic loading of the anchor which may cause cyclic creep which would eventually accumulate to the point of reducing anchor depth and, thus, capacity, and may eventually lead to failure; and (3) cyclic loading and strength reduction of the soil mass by earthquake loading which in some cases can reduce an anchor's capacity under all types of loadings. Anchors subjected to direct application of cyclic loading must be designed utilizing the procedures in the first two categories; all anchors located in earthquake areas must take into account the design procedure of the third category.

Strength Loss During Anchor Cyclic Loading

Some soils, such as very loose, fine-grained granular soils of uniform size (fine sands or coarse silts and some clean oozes), are susceptible to true liquefaction failure when subjected to cyclic loading. Most of these highly susceptible soils are specifically excluded from use of these guidelines because they qualify as hazardous soils according to the definition given in the earlier section on site conditions. Most other soils, however, including very plastic cohesive soils, are subject to some strength loss, especially under extended cyclic loading conditions. This strength loss is related to the development of excess pore pressures which accompany prolonged cyclic loading when drainage/dissipation of these excess pressures is impeded by soils of lower permeability. In general, the denser the soil is (or the more plastic the soil) and the lower the cyclic load, the less susceptible to strength loss is the soil.

As described in the earlier section on cyclic loads, the most critical cyclic loads are characterized in terms of their double-amplitude magnitude and the number of cycles during a period of time t_{cd} in which the excess pressure cannot dissipate and effectively erase the soil's memory of cumulative loading effects. This characterization is either in terms of the total number of cycles at some average uniform magnitude or of a spectral distribution of load magnitudes as a function of number of cycles. For either case, the loading is characterized using previous definitions and Figure 19 to determine t_{cd} from the soil's permeability. The number of load cycles during a period equal to t_{cd} is found, and limiting design bounds as a function of soil type are then established using Figure 20. Figure 20 can be used to find the limiting number of cycles for a given loading or the limiting loading for a given number of cycles. The upper bounds apply to cases where the average quasi-static load, P_{ave}, is one-third or less of the static holding capacity. For the unlikely case where the average quasi-static load is greater than one-third of the static holding capacity, the excess is added singularly to the double-amplitude cyclic

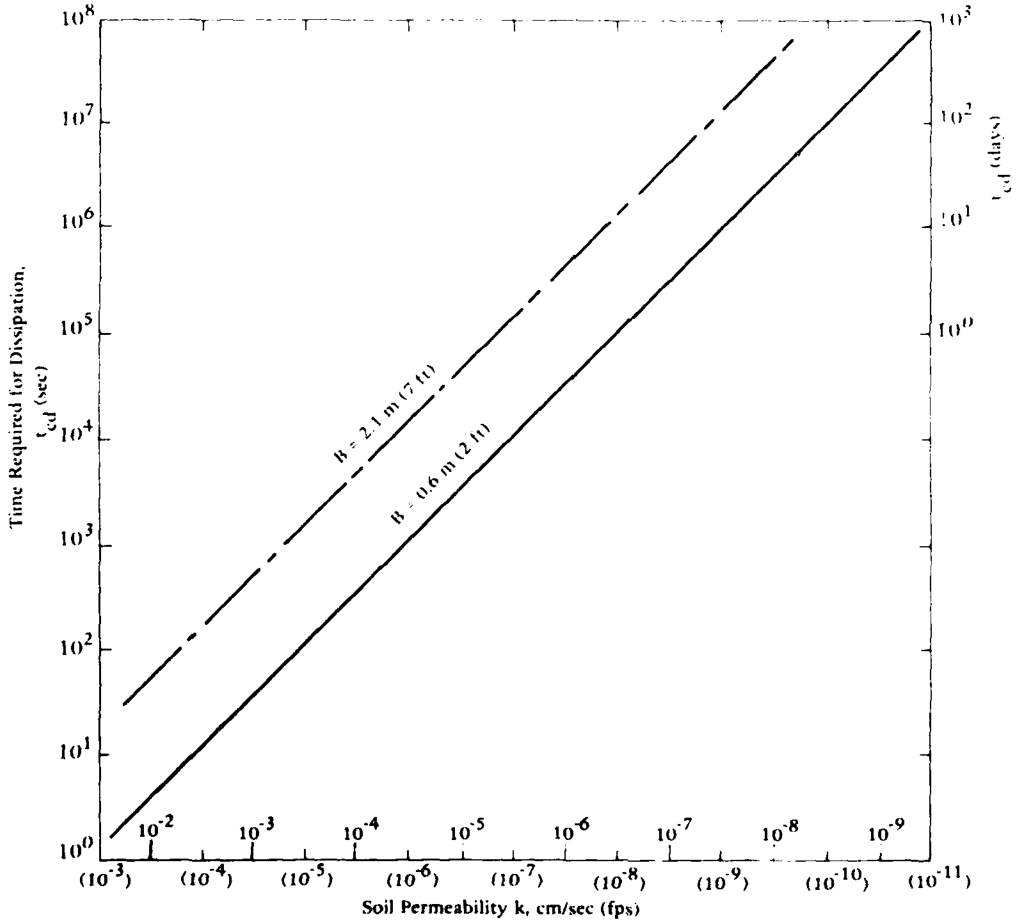


Figure 19. Times required for excess pore pressure redistribution/dissipation.

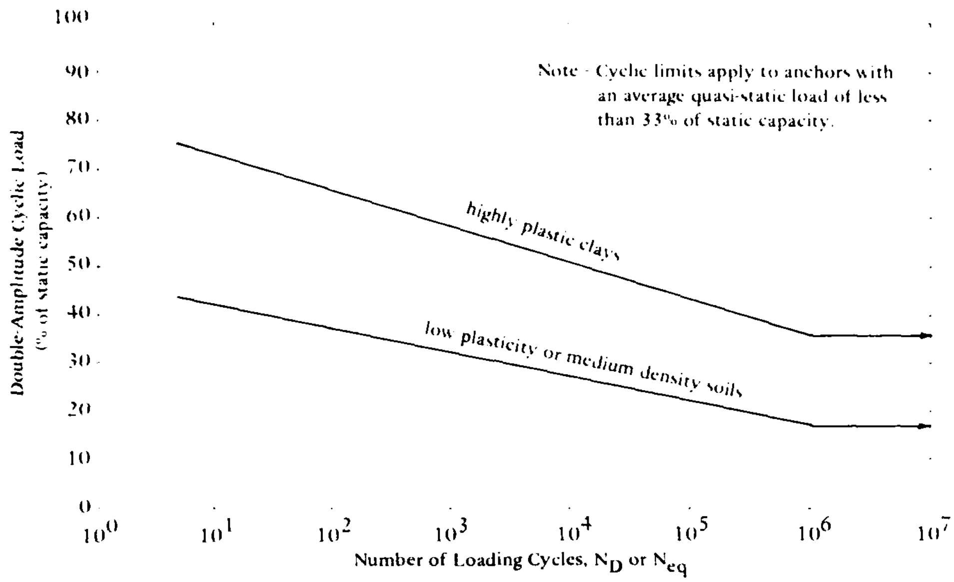


Figure 20. Contours of cyclic capacity without excess pore pressure dissipation.

load prior to using Figure 20. Use of suitable factors of safety is discussed later. The effect of stress history in terms of previous cyclic or impact loads is discussed subsequently in the section on that subject.

Cyclic Creep During Anchor Cyclic Loading

The mechanism leading to cyclic creep of an embedment anchor is not well understood but is known to occur under loading conditions which in some cases are quite safe relative to the criteria for cyclic strength loss presented in the preceding section. For cyclic creep considerations, the number and magnitude of significant loading cycles occurring during the lifetime of an anchor control and should be summarized in spectral or quasi-spectral format. The number of significant loading cycles may not be as large as one would expect. For example, assume a mooring system has a planned 20-year life, is continuously in use, and is subjected to significant wave loading during ten 3-day storms per year; the total number of significant cyclic loads will likely be less than one million.

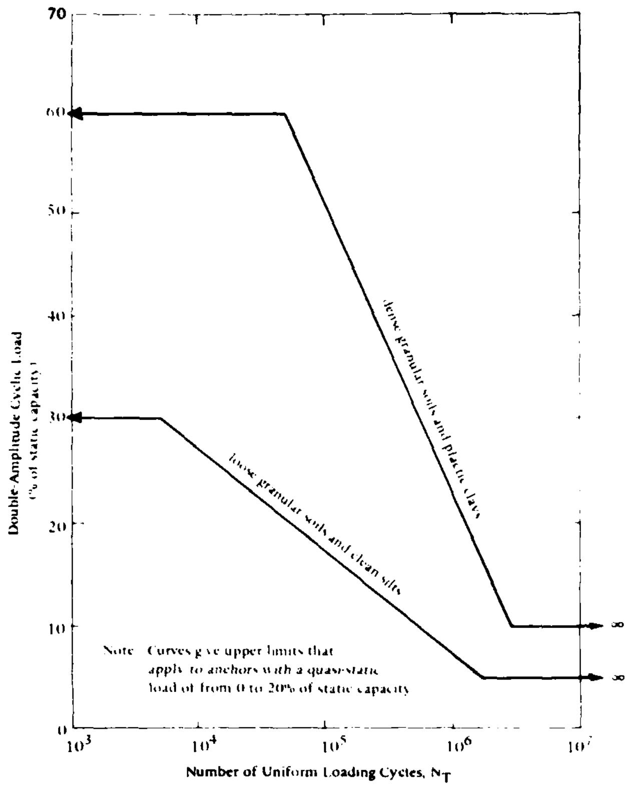


Figure 21. Contours of maximum cyclic loading which will not cause significant cyclic creep.

The established criteria for maximum cyclic loading are presented in spectral format in Figure 21. This figure allows determination of the allowable number of loading cycles for a known double-amplitude cyclic load, or determination of the allowable double-amplitude cyclic load for a known number of loading cycles. Criteria are presented for two categories of soil type. The more restrictive criterion applies to a few sites for which these guidelines are applicable; most such sites are excluded as hazardous sites as defined in an earlier chapter. The criteria established in Figure 21 are applicable to cases where the average quasi-static load, P_{ave}, is less than 20% of quasi-static anchor capacity. For cases where the value of P_{ave} is greater than 20%, that portion above 20% should be singularly added to the double-amplitude cyclic load and the analysis continued. This requirement is quite restrictive for longer life anchor systems subjected to significant, long-term cyclic loading; however, cyclic creep of anchors is not well understood and, until further data are available which show less restrictive criteria to be applicable, this relatively conservative approach is recommended.

Applicable factors of safety are discussed later.

Design for Earthquake Loading

A major earthquake centered within 100 miles of an anchor can temporarily reduce the anchor capacity for all types of loads, but this possibility exists only for relatively clean, granular soils (e.g., sands or coarse silts of uniform size and with few fines - fine silts or clay size particles). Cohesive soils do not lose any significant amount of strength (relative to anchor capacity) in the 30 or less significant loading cycles associated with major earthquakes. As discussed earlier in the section on site survey, a granular soil's susceptibility to strength reduction during an earthquake is primarily a function of its relative density and partially a function of soil depth.

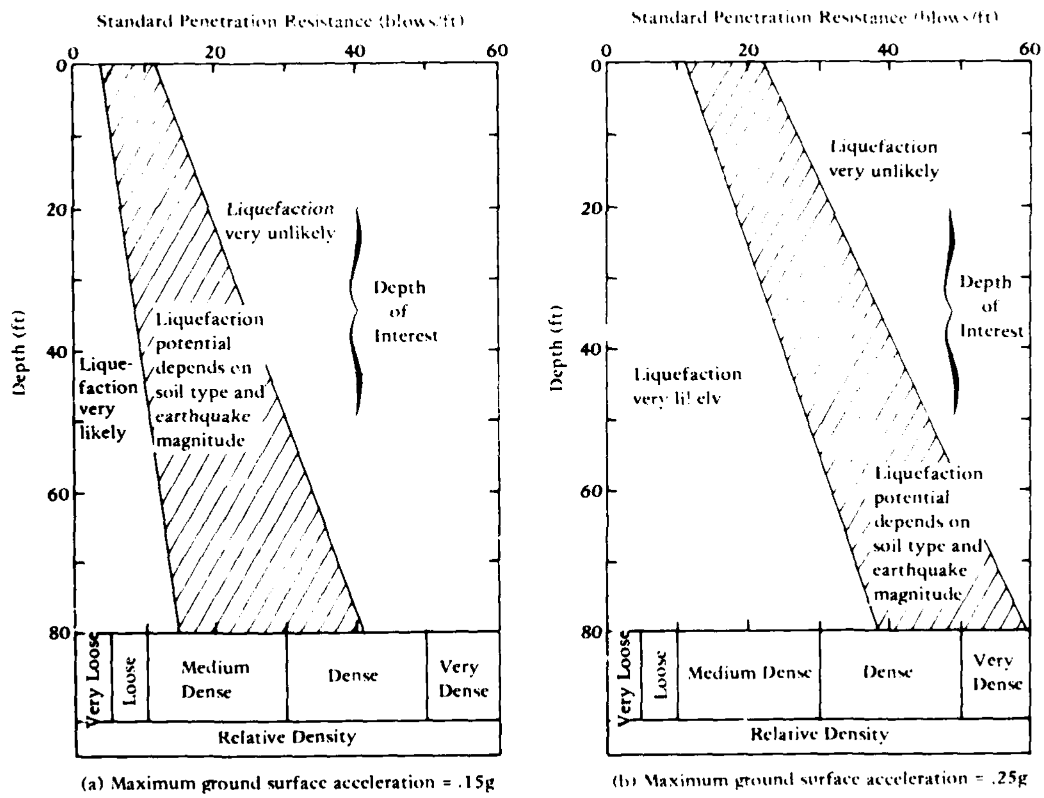


Figure 22. Liquefaction potential profiles for earthquake loading of granular soils (after Seed and Idriss, 1971. Used by permission of ASCE).

The criteria for liquefaction are given on Figure 22 for two peak acceleration levels. Interpolation or limited extrapolation can be used to assess stability at the site in question based on the exact value of peak acceleration determined in the earlier discussion on earthquake loading. Conditions should be assessed at the depth of the keyed anchor and just above it.

If analysis of a site and its expected earthquake indicates that liquefaction is very likely, the site is hazardous to an extent. In this situation, if the anchor is loaded in any manner during the earthquake (such as with a subsurface buoy) or within a time of about $0.2\ t_{cd}$ immediately following the earthquake, the anchor will likely fail. For soils that will liquefy under earthquake loading, the value of t_{cd} is typically quite short – a matter of minutes at most. Situations that classify as potentially liquefiable are also potentially hazardous. Factors of safety relative to the anchor load or anchor capacity are meaningless in this type of earthquake loading as the entire soil mass is in a state of failure when liquefaction occurs. For anchors that are loaded for a significant percentage of the time in areas prone to major earthquakes, site conditions that indicate a potential or likelihood for liquefaction should be avoided.

For applications having a lower consequence of failure, the possibility (typically a low probability over the lifetime of an embedment anchor system) of a major earthquake in the vicinity and the resultant possibility of an anchor failure may be acceptable.

Factors of Safety for Cyclic Loading

As discussed above, the use of a factor of safety relative to anchor loading or soil strength for an earthquake loading is not appropriate for the simplified guideline approach utilized here. For that loading situation, a "potential hazardous condition" approach is applicable.

For design relative to cyclic strength loss and cyclic creep as described earlier in this section, a factor of safety is used to determine allowable load levels. The approach represented quantitatively by Equations 10 and 17 is appropriate. The factor of safety is applied relative to load levels, not to the number of cycles.

The criteria established for design relative to these two mechanisms of potential failure and the procedures utilized incorporate a number of conservative assumptions and "worst-case" values in both situations to determine limiting conditions. As a result, smaller values of factors of safety may be used (values smaller than those suggested in the section entitled Allowable Loads - Factor of Safety). Where values of 1.5 to 3.0 are suggested for a number of quasi-static loading conditions, values on the order of 1.25 to 1.75 relative to cyclic loads for similar conditions would be appropriate.

EFFECT OF LOAD HISTORY

The performance of an anchor is a function of the soil properties surrounding the anchor fluke. These soil properties can vary over time and as a result of forces applied to the soil by the anchor. Thus, load history can be important.

This load history usually directly affects the pore pressure in the soil. Increasing the pore pressure decreases the anchor's capacity under all types of loading. Any increased or excess pore pressure will decrease over time as a result of drainage/dissipation or redistribution. This generally leads to a denser and stronger condition than originally existed. The time required for this dissipation/redistribution of excess pore pressure to occur is a direct function of the soil's permeability, k. Typical values of permeability are listed in Table 5. The time for dissipation/redistribution t_{cd} is also partially a function of anchor size. Typical values can be determined from Figure 19, which shows that values of t_{cd} can vary from a matter of tens of seconds in clean sands to hundreds of days in clays of medium plasticity.

This load history effect has already been included in a minor way earlier in the report. In the section on anchor keying, it was suggested that as much time as possible (at least hours and hopefully days) be allowed between installation of an anchor in cohesive soil and the keying of it. This delay allows dissipation of the excess pore pressures generated in the soil by soil disturbance as the fluke embedded. Dissipation of the excess pore pressure (or even partial dissipation) results in increased soil strength and, thus, a quicker anchor keying action in a shorter distance. This leads directly to a higher capacity anchorage.

Table 5. Typical Values of Soil Permeability (Based in part on data from Hough (1969), Lambe and Whitman (1969), and Mitchell (1976))

Soil Type	Permeability, k (fps)
Uniform Coarse Sand	1×10^{-2}
Uniform Medium Sand	3×10^{-3}
Well-graded Clean Sand	3×10^{-4}
Uniform Fine Sand	1×10^{-4}
Well-graded Silty (dirty) Sand	1×10^{-5}
Uniform Silt	2×10^{-6}
Silty Clay	3×10^{-8}
Low Plasticity Clay (Kaolinite), PI<20	3×10^{-8}
Medium Plasticity Clay (Illite), PI = 20-60	3×10^{-9}
High Plasticity Clay, PI = 60-200	3×10^{-10}
Very High Plasticity Clay, PI>200	3×10^{-11}

The capacity of an anchor will typically increase over its lifetime (assuming it is not overloaded and/or cyclic creep is not large) because drainage of water to relieve excess positive pore water pressure results in denser and stronger soil. However, the increase is small and very dependent on stress history. Thus, this increase is not quantitatively included in the simplified procedures presented here.

For the purposes of these simplified design procedures, load history is addressed only in terms of situations where anchor load capacity is reduced. The expected loading of the anchorage should be examined to determine the most severe periods of service. Several may have to be checked (including both cyclic and impact loadings) in order to determine the worst case. The span of time to be considered in these checks differs for cyclic and impact loads. For cyclic loads, a period of time equal to t_{cd} must be considered. However, for impact loading, a period of 0.5 t_{cd} is all that needs to be considered. Impact loads are discrete events compared to the fairly continuous nature of cyclic loads. Thus, the damaging effects of impact loads are erased/dissipated more rapidly. The value of t_{cd} is determined from Figure 19 in conjunction with Table 5.

The effect of more than one impact load (as defined in the section entitled Design Procedures for Dynamic/Impact Loading) during a time period of 0.5 t_{cd} is determined from the impact reduction factor for loading history R_I where, for cohesive soil,

$$R_I = 0.75^{(n-1)} \tag{18}$$

For granular soils,

$$R_I = 0.5^{(n-1)} \tag{19}$$

where n is the number of impact loads during a time 0.5 t_{cd}.

This factor R_I is used to calculate impact load capacity in Equations 13 and 14. Cyclic load capacity should likewise be reduced by multiplying the value determined from Figure 20 by R_I when more than one impact loading is expected during the time period t_{cd}. As is obvious from the above equations, the influence of several impact loadings can be very large, especially in granular (sand) soils. These reduction factors are not used when addressing cyclic creep.

Cyclic loads occurring during any period of time t_{cd} are considered by summarizing the cyclic loading spectrum into an equivalent number of uniform cyclic loads using the procedure described in the section entitled Cyclic Loads. The resulting number of equivalent uniform cyclic loads, N_{eq}, of magnitude P_{ceq} is then compared to the magnitude required to cause failure (for that number of loading cycles) in Figure 20. The resulting percentage is then used in Figure 23 to determine the cyclic reduction factor for load history, R_c. The effect of this factor on impact loading is determined in Equations 13 and 14. This reduction factor is not used in analysis of cyclic capacity as the influence is already considered in the criteria presented in Figures 20 and 21.

ACKNOWLEDGMENTS

The section on design procedures for impact loading, along with directly related sections in other sections, was based on draft material prepared by Mr. Bruce Douglas and Dr. Don Anderson of Earth Technology, Incorporated (formerly Fugro, Incorporated) in Long Beach, Calif. Major portions of this report are based on earlier reports and procedures developed by fellow researchers at the Naval Civil Engineering Laboratory as indicated by the reference list. This material support, along with the direct technical input of Dr. Dan True and a great deal of help from Mr. Rick Beard in getting this report into final form, are gratefully acknowledged.

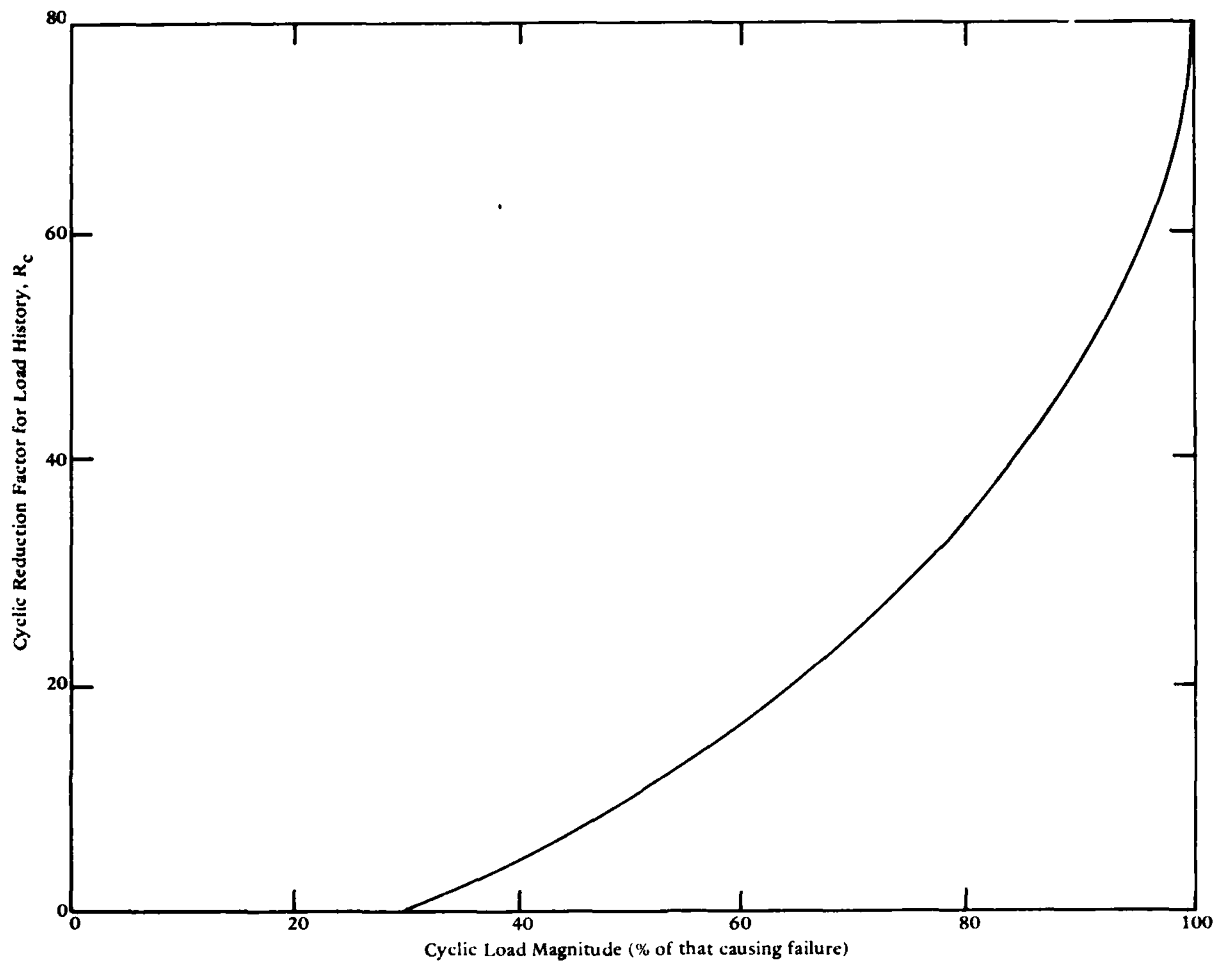


Figure 23. Reduction factor for effect of previous cyclic loading.

REFERENCES

Babineau, P. H., and R. J. Taylor (1976). CEL 10K propellant-actuated anchor operations manual, *Civil Engineering Laboratory*, Technical Memorandum No. M-42-76-3. Port Hueneme, Calif., Apr 1976.

Beard, R. M. (1976). Expendable doppler penetrometer: Interim report, Civil Engineering Laboratory, Technical Note No. N-1435. Port Hueneme, Calif., Apr 1976.

Beard, R. M. (1977). Expendable doppler penetrometer: A performance evaluation, Civil Engineering Laboratory, Technical Report No. R-855. Port Hueneme, Calif., Jul 1977.

Beard, R. M. (1979). Long-term holding capacity of statically loaded anchors in cohesive soils, Civil Engineering Laboratory, Technical Note No. N-1545. Port Hueneme, Calif., Jan 1979.

Beard, R. M. (1980). Holding capacity of plate anchors, Civil Engineering Laboratory, Technical Report No. R-882. Port Hueneme, Calif., Oct 1980.

Castro, G., and S. J. Poulos (1976). "Factors affecting liquefaction and cyclic mobility," Preprints of Liquefaction Problems in Geotechnical Engineering Conference, American Society of Civil Engineers, Philadelphia, Pa., Sep 1976, pp 105-137.

Douglas, B. J. (1978). Effect of rapid loading rates on the holding capacity of direct embedment anchors, Civil Engineering Laboratory, P. O. Report No. 78-M-R420. Port Hueneme, Calif., Oct 1978.

Gouda, Z. M., and D. G. True (1977). Dynamic loading effects on embedment anchor holding capacity – Interim report, Civil Engineering Laboratory, Technical Note No. N-1489. Port Hueneme, Calif., Jul 1977.

Herrmann, H. G., D. A. Raecke, and N. D. Albertsen (1972). Selection of practical seafloor foundation systems. Naval Civil Engineering Laboratory, Technical Report No. R-761, Port Hueneme, Calif., Mar, 1972.

Herrmann, H. G., and W. N. Houston (1976). "Response of seafloor soils to combined static and cyclic loading," in Proceedings of Eighth Offshore Technology Conference, Houston, Tex., May 1976.

Herrmann, H. G., and W. N. Houston (1978). "Behavior of seafloor soils subjected to cyclic loading," in Proceedings of Tenth Offshore Technology Conference, Houston, Tex., May 1978.

Horn, D. R., M. N. Delach, and B. M. Horn (1974). "Physical properties of sedimentary provinces, North Pacific and North Atlantic Ocean," Deep Sea Sediments: Physical and Mechanical Properties, ed. by A. L. Inderbitzen. New York, N.Y., Plenum Press, 1974, pp 417-442.

Hough, B. K. (1969). Basic soils engineering. New York, N.Y., The Ronald Press Company, 1969.

Houston, W. N., and H. G. Herrmann (1980). "Undrained cyclic strength of marine soils," Journal of the Geotechnical Engineering Division, ASCE, vol 106, no. GT6, Jun 1980, pp 691-712. (Paper No. 15508)

Keller, G. H. (1974). "Marine geotechnical properties: Interrelationships and relationships to depth of burial," Deep-Sea Sediments: Physical and Mechanical Properties, ed. by A. L. Inderbitzen. New York, N.Y., Plenum Press, 1974, pp 77-100.

Lambe, T. W., and R. V. Whitman (1969). Soil Mechanics. New York, N.Y., John Wiley and Sons, Inc., 1969.

Lee, H. J., and J. E. Clausner (1979). Seafloor soil sampling and geotechnical parameter determination - Handbook, Civil Engineering Laboratory, Technical Report No. R-873. Port Hueneme, Calif., Sep 1979.

Lee, K. L., and J. A. Focht (1975). "Liquefaction potential at Ekofisk tank in North Sea," Journal of the Geotechnical Engineering Division, ASCE, vol 101, no. GT1, Jan 1975, pp 1-18. (Paper No. 11054)

Malloy, R. J., and P. J. Valent (1978). Acoustic siting and verification of the holding capacity of embedment anchors, Civil Engineering Laboratory, Technical Note No. N-1523. Port Hueneme, Calif., Jul 1978.

McClelland, B. (1969). "Foundations," Handbook of Ocean and Underwater Engineering, ed. by J. J. Myers, C. H. Holm, and R. F. McAllister. New York, N.Y., McGraw-Hill Book Company, 1969, pp 8-98 to 8-134.

McClelland Engineers, Inc. (1976). Deep sea drilling project soils study: Continental margin sites, Consulting Report to University of California Deep Sea Drilling Project. La Jolla, Calif., Jun 1976.

Mitchell, J. K. (1976). Fundamentals of Soil Behavior. New York, N.Y., John Wiley and Sons, Inc., 1976.

Naval Facilities Engineering Command (1971). Design manual - Soil mechanics, foundations and earth structures, NAVFAC Design Manual No. DM-7. Washington, D.C., 1971.

Palo, P. A., and R. L. Webster (1980). Static and dynamic moored tanker response, Journal of the Applied Mechanics Division, ASME, AMD, vol 37, 1980.

Peck, R. B., W. E. Hanson, and T. H. Thornburn (1953). Foundation engineering. New York, N.Y., John Wiley and Sons, 1953.

Rocker, K. (1977). Reduction of embedment anchor capacity due to sediment disturbance, Civil Engineering Laboratory, Technical Note No. N-1491. Port Hueneme, Calif., Jul 1977.

Seed, H. B., and I. M. Idriss (1971). "Simplified procedure for evaluating soil liquefaction potential," Journal of the Soil Mechanics and Foundations Division, ASCE, vol 97, no. SM9, Sep 1971, pp 1249-1273. (Paper No. 8371)

Seed, H. B., I. M. Idriss, and F. W. Kiefer (1969). "Characteristics of rock motions during earthquakes," Journal of the Soil Mechanics and Foundations Division, ASCE, vol 95, no. SM5, Sep 1969, pp 1199-1218. (Paper No. 6783)

Taylor, R. J. (1976). CEL 20K propellant-actuated anchor, Civil Engineering Laboratory, Technical Report No. R-837. Port Hueneme, Calif., Mar 1976.

Taylor, R. J., and P. H. Babineau (1974). Operation manual – Propellant-actuated deep water anchor, Civil Engineering Laboratory, Technical Memorandum No. M-42-74-1. Port Hueneme, Calif., Sep 1974.

Taylor, R. J., D. Jones, and R. M. Beard (1975). Handbook for uplift-resisting anchors, Civil Engineering Laboratory. Port Hueneme, Calif., Sep 1975.

True, D. G. (1975). Penetration of projectiles into seafloor soils, Civil Engineering Laboratory, Technical Report No. R-822. Port Hueneme, Calif., May 1975.

Valent, P. J. (1978). Results of some uplift capacity tests on direct embedment anchors, Civil Engineering Laboratory, Technical Note No. N-1522. Port Hueneme, Calif., Jun 1978.

Wadsworth, J. F. (1976). Anchoring in rock - A preliminary study, Civil Engineering Laboratory, Technical Memorandum No. M-42-76-5. Port Hueneme, Calif., Apr 1976.

Wadsworth, J. F., and R. J. Taylor (1976). CEL 10K propellant-actuated anchor, Civil Engineering Laboratory, Technical Note No. N-1441. Port Hueneme, Calif., Jun 1976.

Wilson, B. W. (1969). Earthquake occurrence and effects in ocean areas, Contract Report CR-69.027, Naval Civil Engineering Laboratory. Port Hueneme, Calif., Feb 1969.

LIST OF SYMBOLS

A	Bearing area of a keyed fluke (ft^2)
A_F	Frontal area of a penetrating anchor fluke (ft^2)
A_s	Side area of a penetrating anchor fluke and piston (ft^2)
B	Fluke width (ft)
c	Undrained shear strength of a cohesive soil (equivalent to s_u as used in these guidelines) (psf)
D	Soil depth of keyed anchor (ft)
D_{pk}	Maximum depth of fluke tip penetration (ft)
f	Correction factor to account for soil disturbance
F_{all}	Allowable (or design) quasi-static load on an anchor (lb)
F_I	Anchor capacity under a given impact loading condition (lb)
F_{Iall}	Allowable (or design) anchor load for a given impact loading condition (lb)
F_s	Factor of safety
F_T	Anchor quasi-static holding capacity (lb)
H	Wave height (ft)
I	Strength influence factor for impact loading
I_F	Inertial factor for impact loading
k	Soil permeability (fps)
L	Fluke length (ft)
ℓ_T	Length of fluke plus piston (ft)
n	Number of impact loads during a time 0.5 t_{cd}
N	Number of waves or loading cycles
$\bar{N}_c$	Short-term anchor holding capacity factor in cohesive soils
N_D	Number of loading cycles in a drainage time period of t_{cd}

N_{eq}	Number of equivalent uniform loading cycles
N_{if}	Number of cycles of magnitude P_{cci} required to cause significant damage
$\bar{N}_q$	Anchor holding capacity factor in granular soils
$\bar{N}_{qI}$	Anchor holding capacity factor for impact loading in granular soils
N_{ref}	Number of loading cycles of magnitude P_{ceq} to cause significant damage
N_{ru}	Number of groups/categories of relatively uniform cyclic loading magnitude
N_T	Total number of significant loading cycles during life of an anchor
OCR	Overconsolidation ratio
p	Effective vertical soil overburden pressure (p' in some texts and equivalent to σ_v', equal to z times γ_b) (psf)
P_{ave}	Average quasi-static load during cyclic loading (lb)
P_{cc}	Double amplitude cyclic load component (lb)
P_{ceq}	Equivalent uniform double-amplitude cyclic load component (lb)
P_I	Impact load (lb)
PI	Plasticity index
P_u	Cyclic anchor load (lb)
R_c	Cyclic reduction factor for loading history
R_I	Impact reduction factor for loading history
S_T	Soil sensitivity; ratio of undisturbed-to-remolded strength of soil (dimensionless)
s_u	Undrained shear strength of cohesive soil (psf)
s_{uI}	Undrained shear strength of cohesive soil under impact loading (psf)
T	Fluke thickness (ft)
t_{cd}	Time required for dissipation or redistribution of most of the excess pore pressures in the vicinity of the anchor fluke (sec)
t_I	Duration of impact load (sec)

v	Anchor penetration velocity (fps)
W_T	In-air weight of anchor fluke and piston (lb)
z	Soil depth to a point of interest (ft)
Δz	Increment of soil depth (ft)
γ_b	Buoyant or submerged unit weight of soil (pcf)
ϕ'	Drained friction angle for granular soil (deg)
ϕ_I	Effective friction angle under impact loading (deg)
ϕ_u'	Friction angle at large displacements (deg)
σ_3	Principal lateral stress (psf)
σ_{3c}	Principal lateral stress during consolidation (psf)

DISTRIBUTION LIST

AFB CESCH, Wright-Patterson; HQ Tactical Air Cmd DEMM (Schmidt) Langley, VA; MAC DET (Col. P. Thompson) Scott, IL; MACOS-XOXC (Col Lee) Scott, IL; Stinfo Library, Offutt NE
ARCTICSUBLAB Code 54, San Diego, CA
ARMY 36th Engineer Group AFVK-C (Capt P Topp), Ft Benning, GA; BMDSC-RE (H. McClellan) Huntsville AL; DAEN-MPE-D Washington DC; *QTRMSTR Scol (Code ATSM-CD) Ft. Lee, VA*
ARMY COASTAL ENGR RSCH CEN Fort Belvoir VA; R. Jachowski, Fort Belvoir VA
ARMY COE Philadelphia Dist. (LIBRARY) Philadelphia, PA
ARMY CORPS OF ENGINEERS MRD-Eng. Div., Omaha NE; Seattle Dist. Library, Seattle WA
ARMY CRREL A. Kovacs, Hanover NH; Library, Hanover NH
ARMY DARCOM Code DRCMM-CS Alexandria VA
ARMY ENG WATERWAYS EXP STA Library, Vicksburg MS
ARMY ENGR DIST. Library, Portland OR
ARMY ENVIRON. HYGIENE AGCY HSE-EW Water Qual Eng Div Aberdeen Prov Grnd MD
ARMY MATERIALS & MECHANICS RESEARCH CENTER Dr. Lenoe, Watertown MA
ARMY MOBIL EQUIP R&D COM Code DRDME-GE (Heavy) Fort Belvoir, VA; DRDME-GS Fuel Tech Br, Ft Belvoir, VA
ARMY TRANSPORTATION SCHOOL Code ATSPO CD-TE Fort Eustis, VA
ARMY-DEPOT SYS COMMAND DRSDS-AI (E N Sandell) Chambersburg PA
ASST SECRETARY OF THE NAVY Spec. Assist Energy (Leonard), Washington, DC; Spec. Assist Submarines, Washington DC
BUREAU OF RECLAMATION Code 1512 (C. Selander) Denver CO
CINCLANT Civil Engr. Supp. Plans. Ofr Norfolk, VA
CINCPAC Fac Engrng Div (J44) Makalapa, HI
CNM Code MAT-04, Washington, DC; MAT-0718, Washington, DC; NMAT - 044, Washington DC
CNO Code NOP-964, Washington DC; Code OP 323, Washington DC; Code OP 987 Washington DC; Code OP-413 Wash, DC; Code OPNAV 09B24 (H); Code OPNAV 22, Wash DC; Code OPNAV 23, Wash DC; OP-23 (Capt J.H. Howland) Washinton, DC; OP987J (J. Boosman), Pentagon
COMCBPAC Operations Off, Makalapa HI
COMDEVGRUONE CMDR San Diego, CA
COMFLEACT, OKINAWA PWO, Kadena, Okinawa
COMNAVBEACHPHIBREFTRAGRU ONE San Diego CA
COMNAVMARIANAS Code N4, Guam
COMSUBDEVGRUONE Operations Offr, San Diego, CA
NAVSURFLANT Norfolk, VA
NAVSURFPAC Code N-4, Coronado
COMOPTEVFOR CMDR, Norfolk, VA; Code 705, San Diego, CA
DEFENSE INTELLIGENCE AGENCY DB-4C1 Washington DC
DEFFUELSUPPCEN DFSC-OWE (Grafton) Alexandria, VA; DFSC-OWE (Term Engrng) Alexandria, VA
DLSIE Army Logistics Mgt Center, Fort Lee, VA
DNA STTL, Washington DC
DOE (R Cohen) Div of Ocean Energy Sys Cons & Solar Energy Wash,
DTIC Defense Technical Info Ctr/Alexandria, VA
DTNSRDC Anna Lab, Code 2724 (D Bloomquist) Annapolis, MD; Anna Lab, Code 2842 (Mr. Sherman) Annapolis, MD
DTNSRDC Code 4111 (R. Gierich), Bethesda MD
GSA Assist Comm Des & Cnst (FAIA) D R Dibner Washington, DC
HCU ONE CO, Bishops Point, HI
KWAJALEIN MISRAN BMDSC-RKL-C
LIBRARY OF CONGRESS Washington, DC (Sciences & Tech Div)
MARINE CORPS BASE PWD - Maint. Control Div. Camp Butler, Kawasaki, Japan; PWO, Camp S. D. Butler, Kawasaki Japan
MARINE CORPS HQS Code LM-2, Washington, DC
MCAS Facil. Engr. Div. Cherry Point NC; CO, Kaneohe Bay HI; Code S4, Quantico VA; Facs Maint Dept - Operations Div, Cherry Point

MCDEC M&L Div Quantico VA; NSAP REP, Quantico VA
MCRD PWO, San Diego Ca
MILITARY SEALIFT COMMAND Washington DC
NAF PWO Sigonella Sicily; PWO, Atsugi Japan
NALF OINC, San Diego, CA
NARF Code 100, Cherry Point, NC; Equipment Engineering Division (Code 61000), Pensacola, FL
NAS Dir. Util. Div., Bermuda; ENS Buchholz, Pensacola, FL; PWD - Engr Div, Oak Harbor, WA; PWD Maint. Div., New Orleans, Belle Chasse LA; PWD, Code 1821H (Pfankuch) Miramar, SD CA; PWD, Willow Grove PA; PWO Belle Chasse, LA; PWO Key West FL; PWO Whiting Fld, Milton FL; PWO, Glenview IL; ROICC Key West FL; SCE Norfolk, VA; SCE, Barbers Point HI; Shore Facil Ofr Norfolk, VA
NATL BUREAU OF STANDARDS Kovacs, Washington, D.C.
NATL RESEARCH COUNCIL Naval Studies Board, Washington DC
NAVACT PWO, London UK
NAVAEROSPREGMEDCEN SCE, Pensacola FL
NAVAIRDEVCEN Code 813, Warminster PA
NAVCOASTSYSCEN Code 719, Panama City, FL; Code 772 (C B Koesy) Panama City FL
NAVCOASTSYSTCTR CO, Panama City FL; Code 715 (J Quirk) Panama City, FL; Code 715 (J. Mittleman) Panama City, FL; Library Panama City, FL
NAVCOMMAREAMSTRSTA PWO, Wahiawa HI; SCE Unit 1 Naples Italy
NAVCOMMSTA Code 401 Nea Makri, Greece; PWD - Maint Control Div, Diego Garcia Is.; PWD-ENG Div Dir, Washington, DC; PWO, Exmouth, Australia
NAVCONSTRACEN Code 00U15, Port Hueneme CA; Curriculum Instr. Stds Offr, Gulfport MS
NAVEDTRAPRODEVCEN Technical Library, Pensacola, FL
NAVELEXSYSCOM Code PME 124-61, Washington, DC; PME 124-612, Wash DC
NAVEODFAC Code 605, Indian Head MD
NAVFAC PWO, Centerville Bch, Ferndale CA
NAVFAC PWO, Lewes DE
NAVFACENGCOM Code 032 (Essoglou) Alexandria, VA; Code 043 Alexandria, VA; Code 044 Alexandria, VA; Code 0453 (D. Potter) Alexandria, VA; Code 0453C, Alexandria, VA; Code 0454B Alexandria, Va; Code 04A1 Alexandria, VA; Code 04B3 Alexandria, VA; Code 0632, Alexandria, VA; Code 100 Alexandria, VA; Code 1002B (J. Leimanis) Alexandria, VA; Code 1113 (M. Carr) Alexandria, VA; Code 1113 (T. Stevens) Alexandria, VA; Morrison Yap, Caroline Is.
NAVFACENGCOM - CHES DIV. Code 405 Wash, DC; Code 407 (D Scheesele) Washington, DC; Code FPO-1C Washington DC; Code FPO-1E, Wash. DC; FPO-1 Wash, DC; FPO-1 Washington, DC; FPO-1EA5 Washington DC; FPO-1P/1P3 Washington, DC
NAVFACENGCOM - LANT DIV. Code 10A, Norfolk VA; Eur. BR Deputy Dir, Naples Italy; European Branch, New York; RDT&ELO 102, Norfolk VA
NAVFACENGCOM - NORTH DIV. (Boretsky) Philadelphia, PA; CO; Code 04 Philadelphia, PA; Code 09P Philadelphia PA; Code 1028, RDT&ELO, Philadelphia PA; ROICC, Contracts, Crane IN
NAVFACENGCOM - PAC DIV. CODE 09P PEARL HARBOR HI; Code 2011 Pearl Harbor, HI; Code 402, RDT&E, Pearl Harbor HI; Commander, Pearl Harbor, HI
NAVFACENGCOM - SOUTH DIV. Code 90, RDT&ELO, Charleston SC
NAVFACENGCOM - WEST DIV. AROICC, Contracts, Twentynine Palms CA; Code 04B San Bruno, CA; 09P/20 San Bruno, CA; RDT&ELO Code 2011 San Bruno, CA
NAVFACENGCOM CONTRACT AROICC, Point Mugu CA; Dir, Eng. Div., Exmouth, Australia; Eng Div dir, Southwest Pac, Manila, PI; OICC, Southwest Pac, Manila, PI; OICC/ROICC, Balboa Canal Zone; ROICC AF Guam; ROICC, Diego Garcia Island; ROICC, Keflavik, Iceland; ROICC, NAS, Corpus Christi, TX; ROICC, Pacific, San Bruno CA
NAVFORCARIB Commander (N42), Puerto Rico
NAVNUPWRU MUSE DET Code NPU-30 Port Hueneme, CA
NAVOCEANO Library Bay St. Louis, MS
NAVOCEANSYSCEN Code 41, San Diego, CA; Code 4473 Bayside Library, San Diego, CA; Code 4473B (Tech Lib) San Diego, CA; Code 52 (H. Talkington) San Diego CA; Code 5204 (J. Stachiw), San Diego, CA; Code 5214 (H. Wheeler), San Diego CA; Code 5221 (R.Jones) San Diego Ca; Code 5311 (Bachman) San Diego, CA; Hawaii Lab (R Yumori) Kailua, HI; Hi Lab Tech Lib Kailua HI
NAVPETRES Director, Washington DC
NAVPGSCOL C. Morers Monterey CA; Code 61WL (O. Wilson) Monterey CA; E. Thornton, Monterey CA

NAVPHIBASE CO, ACB 2 Norfolk, VA; COMNAVBEACHGRU TWO Norfolk VA; Code S3T, Norfolk VA; Dir. Amphib. Warfare Brd Staff, Norfolk, VA; Harbor Clearance Unit Two, Little Creek, VA; SCE, Coronado, San Diego CA

NAVREGMEDCEN Chief of Police, Camp Pendleton CA; SCE (D. Kaye); SCE, Guam

NAVREGMEDCEN SCE, Yokosuka, Japan

NAVSCOLCECOFF C35 Port Hueneme, CA

NAVSCSOL PWO, Athens GA

NAVSEASYSCOM Code C132 (Mr. J. Peters) Washington, DC; Code OOC-D, Washington, DC; Code PMS 395 A 3, Washington, DC; Code PMS 395 A2, Washington, DC; Code SEA OOC Washington, DC; PMS-395 A1, Washington, DC; PMS395-A3, Washington, DC; SEA 04E (L. Kess) Washington, DC

NAVSECGRUACT PWO, Adak AK

NAVSHIPREPFAC Library, Guam; SCE Subic Bay

NAVSHIPYD Bremerton, WA (Carr Inlet Acoustic Range); Code 202.4, Long Beach CA; Code 380, Portsmouth, VA; Code 410, Mare Is., Vallejo CA; Code 440 Portsmouth NH; Code 440, Puget Sound, Bremerton WA; Tech Library, Vallejo, CA

NAVSTA CO Naval Station, Mayport FL; CO Roosevelt Roads P.R. Puerto Rico; Code 4, 12 Marine Corps Dist, Treasure Is., San Francisco CA; Engr. Dir., Rota Spain; Long Beach, CA; Maint. Div. Dir Code 531, Rodman Canal Zone; PWD (LTJG.P.M. Motolenich), Puerto Rico; PWO Pearl Harbor, HI; PWO, Keflavik Iceland; PWO, Mayport FL; SCE, Guam; SCE, Subic Bay, R.P.; Security Offr, San Francisco, CA

NAVSUPPACT LTJG McGarrah, SEC, Vallejo, CA

NAVSUPPFAC PWD - Maint. Control Div, Thurmont, MD

NAVSURFWPNCEN G-52 (Duncan) Dahlgren, VA

NAVTECHTRACEN SCE, Pensacola FL

NAVWPNCEN Code 2636 (W. Bonner), China Lake CA; Code 3803 China Lake, CA

NAVWPNSTA Code 092, Colts Neck NJ

NAVWPNSTA PW Office (Code 09C1) Yorktown, VA

NAVWPNSTA PWD - Maint. Control Div., Concord, CA; PWD - Supr Gen Engr, Seal Beach, CA; PWO, Charleston, SC; PWO, Seal Beach CA

NAVWPNSUPPCEN Code 09 Crane IN

NCTC Const. Elec. School, Port Hueneme, CA

NCBC Code 10 Davisville, RI; Code 15, Port Hueneme CA; Code 155, Port Hueneme CA; Code 156, Port Hueneme, CA

NCR 20, Code R70; 20, Commander

NMCB 74, CO; FIVE, Operations Dept; Forty, CO; THREE, Operations Off.

NOAA (Dr. T. Mc Guinness) Rockville, MD; Library Rockville, MD

NORDA Code 410 Bay St. Louis, MS; Code 440 (Ocean Rsch Off) Bay St. Louis MS; Code 500, (Ocean Prog Off-Ferer) Bay St. Louis, MS

NRL Code 5800 Washington, DC; Code 5843 (F. Rosenthal) Washington, DC; Code 8441 (R.A. Skop), Washington DC

NROTC J.W. Stephenson, UC, Berkeley, CA

NSC Code 44 (Security Officer) Oakland, CA; Code 54.1 (Wynne), Norfolk VA

NSD SCE, Subic Bay, R.P.

NUCLEAR REGULATORY COMMISSION T.C. Johnson, Washington, DC

NUSC Code 131 New London, CT; Code 332, B-80 (J. Wilcox) New London, CT; Code EA123 (R.S. Munn), New London CT; Code TA131 (G. De la Cruz), New London CT

OFFICE SECRETARY OF DEFENSE OASD (MRA&L) Dir. of Energy, Pentagon, Washington, DC

ONR (Scientific Dir) Pasadena, CA; Central Regional Office, Boston, MA; Code 481, Bay St. Louis, MS; Code 485 (Silva) Arlington, VA; Code 700F Arlington VA

PHIBCB 1 P&E, San Diego, CA; 1, CO San Diego, CA; 1, CSWC D Wellington, San Diego, CA

PMTC Code 3144, (E. Good) Point Mugu, CA; Code 3331 (S. Opatowsky) Point Mugu, CA; EOD Mobile Unit, Point Mugu, CA; Pat. Counsel, Point Mugu CA

PWC CO Norfolk, VA; CO, (Code 10), Oakland, CA; CO, Great Lakes IL; CO, Pearl Harbor HI; Code 10, Great Lakes, IL; Code 120, Oakland CA; Code 120C, (Library) San Diego, CA; Code 128, Guam; Code 154, Great Lakes, IL; Code 200, Great Lakes IL; Code 220.1, Norfolk VA; Code 30C, Norfolk, VA; Code 30C, San Diego, CA; Code 400, Great Lakes, IL; Code 400, Pearl Harbor, HI; Code 400, San Diego, CA; Code 420, Great Lakes, IL; Code 420, Oakland, CA; Code 505A (H. Wheeler); Code 600 (Utilities Dept) Norfolk, VA; Code 700, Great Lakes, IL; Code 700, San Diego, CA

SPCC PWO (Code 120) Mechanicsburg PA

SUPANX PWO, Williamsburg VA
UCT TWO OIC, Norfolk, VA; OIC, Port Hueneme CA
US DEPT OF INTERIOR Bur of Land Mngmnt Code 583 (T F Sullivan) Washington, DC
US GEOLOGICAL SURVEY Off. Marine Geology, Piteleki, Reston VA
US NAVAL FORCES Korea (ENJ-P&O)
USCG (G-MP-3/USP/82) Washington Dc: (Smith), Washington, DC; G-ECV (C E Smith) Washington, DC; G-EOE-2/61 (Espinshade), Washington, DC; G-EOE-4 (T Dowd), Washington, DC; Gulf Strike Team, Bay St. Louis, MS; Lant Strike Team Elizabeth City, NC, Pac Strike Team, Hamilton AFB, C
USCG R&D CENTER CO Groton, CT; D. Motherway, Groton CT; S Rosenberg, Groton, C
USDA Forest Service Reg 3 (R. Brown) Albuquerque, NM; Forest Service, San Dimas, CA
USNA Civil Engr Dept (R. Erchyl) Annapolis MD; Ocean Sys. Eng Dept (Dr. Monney) Annapolis, MD; PWD Engr. Div. (C. Bradford) Annapolis MD
WATER & POWER RESOURCES SERVICE (Smoak) Denver, CO
NUSC Library, Newport, RI